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Design of Onsite Wastewater Treatment Systems for Nonpotable Water Reuse at Safety Roadside Rest Area Facilities

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California Department of Transportation
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15. Abstract

This manual contains technical data for the design, as well as some limited information on the operation and maintenance, of various OWTSs applicable to Caltrans SRRA facilities, with a specific focus on SRRA nonpotable water recycling systems. Water recycling systems may be used to collect drainage water from public restrooms, treat the water to the quality required for unrestricted reuse, and return the water solely for toilet and urinal flushing. Wastewater is generated from activities that include hand washing, toilet flushing, urinal flushing, and restroom maintenance. Depending on the type of plumbing fixtures used, average water use can range from about 1.28 to 3.5 gallons per person per use (gal/use), with about 90 percent of the water used for toilet and urinal flushing. The onsite water recycling system designs described are expected to reduce total indoor water use by 75 to 90 percent.

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ACRONYMS AND ABBREVIATIONS

%	percent
§	Section
A	Area
ABR	anaerobic baffled reactor
APMP	advanced protection management plan
APS	advanced planning study
avg	average
BOD	biochemical oxygen demand (5 day)
BTPT	biological treatment process tank
BWF	backwashing filter
BWFDT	backwash filter dosing tank
BWFET	backwash filter effluent tank
C	Celsius
$CaCO_3$	calcium carbonate
Caltrans	California Department of Transportation
cBOD	carbonaceous biochemical oxygen demand (5 day)
CCR	California Code of Regulations
CCT	chlorine contact tank
CHP	California Highway Patrol
Cl_2	chlorine
Clean Water Act	Porter-Cologne Act
COD	chemical oxygen demand
C_RT	product of residual chlorine concentration and time
cu	color unit (platinum-cobalt)
CU	control unit
DDW	Division of Drinking Water
ES	effective size
F	Fahrenheit
EB	eastbound

EPA	United States Environmental Protection Agency
Fe	iron
General Order	SWRCB Order WQ 2014-0153-DWQ – General Waste Discharge Requirements For Small Domestic Wastewater Treatment Systems
H_2S	hydrogen sulfide
HDPE	high-density polyethelene
HET	high-efficiency toilet
Hg	mercury
HLA	high-level alarm
HLR	hydraulic loading rate
HOCl	hypochlorous acid
hp	horsepower
HRT	hydraulic retention time
HVAC	heating, ventilation, and air conditioning
HWL	high water level
ICB	instrumentation and control building
LAGON	lag pump on set point
LAMP	local agency management program
LCP	local control panel
LEED	leadership in energy and environmental design
LLA	low-level alarm
LS	lift station
LWL	low water level
MBR	membrane bioreactor
MDS	monthly data summary
Mn	manganese
MPN	most probable number
MRD	most recent day
MRH	most recent hour
MRM	most recent month

NaOCl	sodium hypochlorite
NB	northbound
NCl_3	nitrogen trichloride
NH_2Cl	monochloramine
NH_4^+-N	ammonium nitrogen
$NHCl_2$	dichloramine
NTU	nephelometric turbidity unit
NR	not removed
O&M	operations and maintenance
OCl^-	hypochlorite
OCT	ozone contact tank
ODT	ozone dosing tank
ORP	oxidation-reduction potential
OWTS	onsite wastewater treatment system
PBF	Advantex packed-bed filter
PERC	Plumbing Efficiency Research Coalition
PFU	plaque forming unit
PLC	programmable logic controller
POFF	pump off set point / lead pump off set point
PON	pump on set point / lead pump on set point
PT	pressure tank
pt-co	platinum-cobalt (color unit)
PVC	polyvinyl chloride
PW	product water
PWPT	product water pressure tank
PWST	product water storage tank
Q	flowrate
$Q_{fut,summer}$	expected future summer-season flowrate
RO	redundant off
RV	recreational vehicle
RWQCB	Regional Water Quality Control Board

SB	southbound
SCADA	supervisory control and data acquisition
sCOD	soluble chemical oxygen demand
SLO	San Luis Obispo
SPT	solids process tank
SRRA	Safety Roadside Rest Area
SRT	solids retention time
SSFDT	slow sand filter dosing tank
SSF	slow sand filter
State	State of California
SU	standard units
SWRCB	(California) State Water Resources Control Board
TDS	total dissolved solids
THM	trihalomethanes
TKN	total Kjeldahl nitrogen
TMDL	total maximum daily load
TN	total nitrogen
TON	total oxidized nitrogen, i.e., total nitrite and nitrate
TSS	total suspended solids
UAF	upflow anaerobic filter
UC	uniformity coefficient
U.S.	United States
USCT	upflow sludge contact tank
UV	ultraviolet
VFA	volatile fatty acids
WB	westbound
WDR	waste discharge requirement

ABBREVIATIONS FOR UNITS OF MEASURE

d	day
d/wk	days per week
flush/pe	flush per person
ft	foot (feet)
ft^2	square foot/feet
ft^2/ft^3	square feet per cubic foot
ft^3/h	cubic feet per hour
ft^3/min	cubic feet per minute
g	gram
g/d	gram per day
g/h	gram per hour
g/filter	gram per filter
g/pe	gram per person
gal	gallon
gal/d	gallons per day
gal/y	gallons per year
gal/flush	gallons per flush
gal/min	gallons per min
gal/pe	gallons per person
gal/capita•d	gallons per capita per day
gal/d•filter	gallons per day per filter
gal/ft^2•d	gallons per square foot per day
gal/min ft^2	gallons per minute per square foot
gal/use	gallons per use
h	hour
h/d	hours per day
hp	horsepower
in	inch
in Hg	inches of mercury
in^2	square inch

kg	kilogram=1000 grams
kg/d	kilogram per day
kW	kilowatt
kWh	kilowatt hour
kWh/d	kilowatt hour per day
L	liter
lb	pound
lb/in.^{2}	pounds per square inch
L/gal	liters per gallon
L/d	liters per day
m	meter
m^2/m^3	square meters per cubic meter
mA	milliamp
mm	millimeter
mo	month
m/h	meter per hour
min	minute
mg	milligram
mg/g	milligrams per gram
mg/L	milligram per liter
mg-min/L	milligrams per minute per liter
mg N/L	milligrams of nitrogen per liter
min	minute
mL	milliliter
Mgal	million gallons
Mgal/y	million gallons per year
mo	month
MPN/100 mL	most probable number per 100 milliliters
nm	nanometer
pe/d	person per day
pe/veh	person per vehicle

s	second
s/min	seconds per minute
veh/d	vehicles per day
V	volt
wk	week
y	year

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1.0 INTRODUCTION

The California Department of Transportation (Caltrans) is responsible for the design and operation of Safety Roadside Rest Areas (SRRAs) throughout the State of California (State). Onsite wastewater treatment systems (OWTSs) are necessary because of the remote nature of SRRA facilities, where connection to a municipal sanitary sewer system is not practical. To address this need, a number of studies have been used to characterize the unique wastewater streams from SRRAs and to validate the effectiveness of new treatment technologies:

This manual contains technical data for the design, as well as some limited information on the operation and maintenance, of various OWTSs applicable to Caltrans SRRA facilities, with a specific focus on SRRA nonpotable water recycling systems. These water recycling systems are used to collect drainage water from public restrooms, treat the water to the quality required for unrestricted reuse, and return the water solely for toilet and urinal flushing. Wastewater is generated from activities that include hand washing, toilet flushing, urinal flushing, and restroom maintenance. Depending on the type of plumbing fixtures used, average water use can range from about 1.28 to 3.5 gallons per person per use (gal/use), with about 90 percent of the water used for toilet and urinal flushing. The onsite water recycling system designs described are expected to reduce total indoor water use by 75 to 90 percent.

Section 1 comprises background information on the design and operation of OWTS at SRRAs as outlined in Chart 1-1.

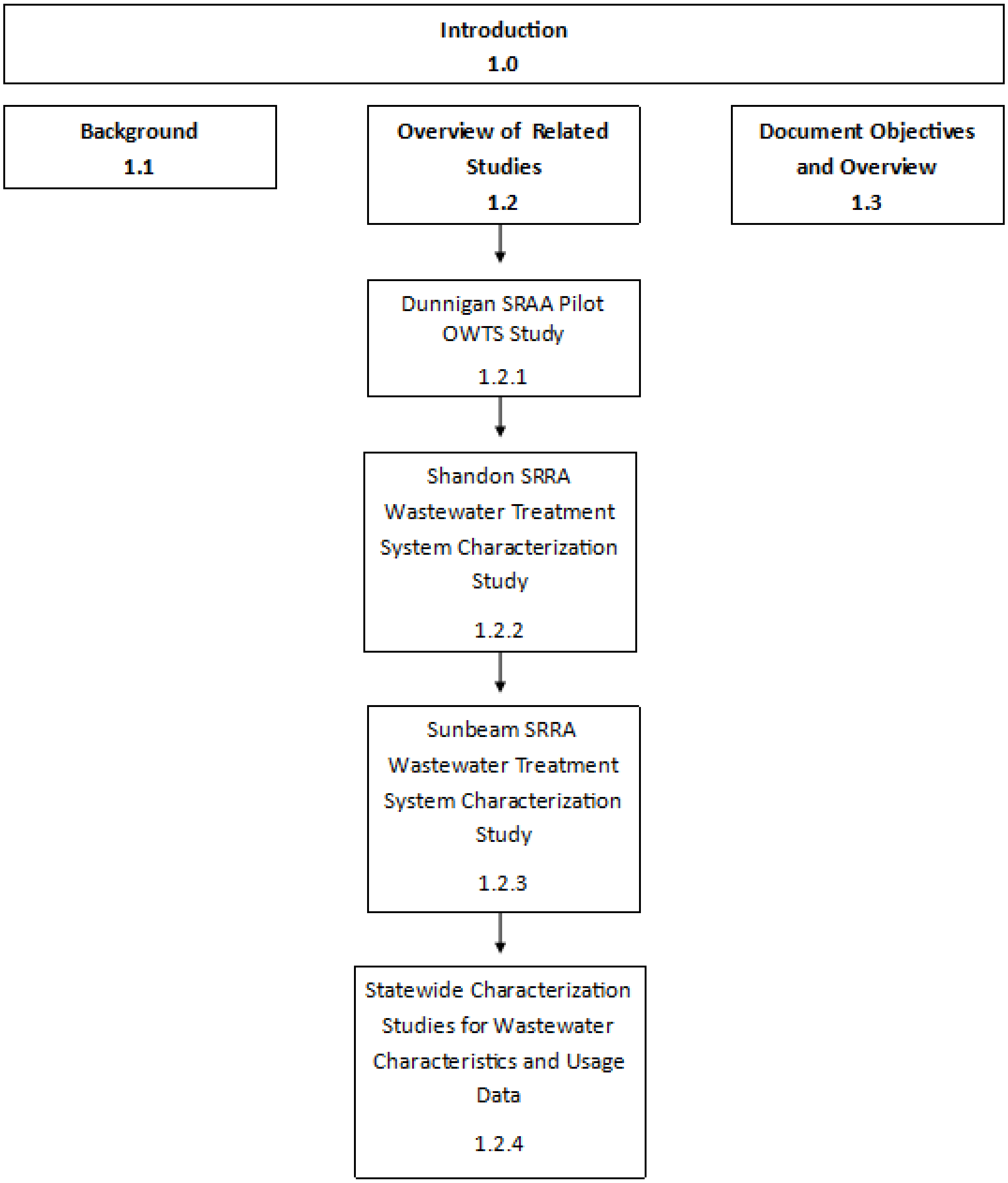

Chart 1-1
Background of design and operation of OWTS at SRRAs

1.1 BACKGROUND

The previous SRRA wastewater systems, which generally discharged to the soil and indirectly to groundwater, ranged from septic systems and ponds to, more recently, advanced nutrient removal systems. The State Water Resources Control Board

(SWRCB) recently adopted a new policy related to OWTSs (discussed in Sec. 2.2) that requires improved treatment of discharges and protection of groundwater resources.

In general, the previously installed technologies used for wastewater treatment at SRRAs were developed in the 1800s. The prior systems were designed to remove and partially digest bulk solids in a septic tank, and then discharge the effluent to subsurface effluent dispersal systems. Since the early 2000s, increased regulatory action has been undertaken to further protect California groundwater and surface water resources. Some Regional Water Quality Control Boards (RWQCBs) have required nitrogen removal to concentrations at or near the drinking water limit in several recent SRRA renovation projects. The previous wastewater systems were not designed for nitrogen removal and could not meet these requirements. Thus, the wastewater treatment systems required to meet modern discharge limits are much more complex than the previous septic tank and pond systems.

1.2 OVERVIEW OF RELATED STUDIES

Because of increasing traffic, most existing onsite wastewater systems require replacement and/or must be upgraded to comply with current environmental requirements. Because of the unique wastewater streams from SRRAs, off-the-shelf designs were found to not be suitable in all circumstances and specialized treatment facilities are sometimes needed (Caltrans, 2010). Therefore, Caltrans has conducted various characterization studies to evaluate the performance of existing and pilot systems at SRRAs throughout the State.

1.2.1 Dunnigan SRRA Pilot OWTS Study

The pilot study was conducted from May 2013 to December 2015 to evaluate multiple new technologies for primary treatment, secondary treatment, tertiary filtration, primary effluent filtration, sensors, controls, and disinfection (described further in Sec. 2.2) (Caltrans, 2013; Document No. CTSW-PL-13-305.02.1.) The purpose of the pilot study was to characterize and benchmark the performance of off-the-shelf and customized wastewater treatment processes to improve the performance, reliability, and maintainability of SRRA wastewater systems throughout the State of California. Data were collected to verify the performance of innovative technologies that have the potential to improve protection of human health and the environment. Routine sampling of each treatment component and challenge testing on disinfection units were conducted to make credible operating data available and to enable the data to be widely distributed and accepted by regulatory agencies and others.

1.2.2 Shandon SRRA Wastewater Treatment System Characterization Study

To obtain a waste discharge permit for the Shandon SRRA, Caltrans developed an advanced wastewater treatment system design to meet new regulatory standards and leadership in energy and environmental design (LEED) criteria, and to reduce operation and maintenance costs (Harold Leverenz and Caltrans Water and Wastewater Branch, 2012). The wastewater treatment system includes a septic tank, recirculation tank and pump system, gravel filter, and subsurface-flow anoxic wetland for passive nitrogen removal. The OWTS is designed to treat the water to a quality deemed safe for environmental discharge. The design is adaptable to conditions found at most Caltrans SRRAs. Monitoring is performed to characterize the wastewater stream, validate the treatment system effectiveness, and confirm compliance with RWQCB requirements. The data that have been collected at the Shandon SRRA are used to better design future wastewater treatment systems, specifically, where nitrogen limits apply.

1.2.3 Sunbeam SRRA Wastewater Treatment System Characterization Study

The wastewater treatment system at the Sunbeam SRRA was designed to (1) produce an effluent low in total nitrogen to meet regulatory requirements, (2) minimize energy use, and (3) simplify operation and maintenance needs (Caltrans, 2010). The wastewater from the facility's comfort stations is currently treated in an OWTS that consists of a settling tank, recirculating sand filter, and wetlands. The result of this technology is clean water that can be reused for irrigation purposes. The facility has been able to significantly reduce the amount of potable water used for landscaping by simply reusing its treated wastewater for irrigation. Although the treatment system makes use of processes that are mostly passive, the operation and maintenance activities were evaluated for long-term performance.

1.2.4 Statewide Characterization Studies for Wastewater Characteristics and Usage Data

A statewide-study was conducted from December 2010 through September 2012 to collect background data and evaluate performance of OWTS equipment and processes at 11 SRRAs. Study sites were located at the Buckman Springs, Dunnigan-northbound (NB), Dunnigan-southbound (SB), Elk Horn, Erreca-NB, Erreca-SB, O'Brien, Lakehead, Shandon, Sunbeam-eastbound (EB), Sunbeam-westbound (WB), Trinidad-NB, and Trinidad-SB SRRAs (Caltrans,2012; Document No. CTSW-RT-12-233.03.1).

The OWTS systems evaluated incorporate technologies such as low-flow water fixtures, urine diversion, biological sand filtration, anoxic-treatment wetlands, drip irrigation, alternative soil dispersal systems, and evaporation ponds. Some of these systems use passive treatment operations that require only pumps with appropriate controls to operate, minimize maintenance, and reduce life cycle costs. The performance of these technologies was evaluated to develop improved designs that meet environmental requirements and to serve as a basis of construction of future onsite wastewater treatment systems at other SRRA sites.

Sites that use conventional septic tanks were also evaluated to characterize the septic tank effluent quality. To further understand wastewater characteristics and system loading, data loggers were installed to collect data on SRRA usage. Installations included (1) people counters to measure the number of persons that enter and exit each men's restroom; (2) data loggers on flow meters to measure influent flow to each restroom and to each treatment component; (3) vehicle counters to measure the number of vehicles that enter the SRRA, and (4) sludge and scum meters to measure sludge and scum levels within the primary tanks.

1.3 DOCUMENT OBJECTIVES AND OVERVIEW

The objective of this document is to provide procedures for the design of various OWTSs applicable to Caltrans SRRA facilities. The information presented in each section is summarized in Table 1-1.

Table 1-1
Document organization

Section	Description
Section 1—Introduction	Provides background information on the water and wastewater systems used at Caltrans SRRA facilities and water recycling pilot studies.
Section 2—Overview Of Onsite Wastewater Treatment Systems	Provides an overview of the components of OWTSs, describes preliminary system configurations, and summarizes regulatory concerns related to onsite wastewater treatment.
Section 3—SRRA Wastewater Characteristics	Provides a procedure for conducting site evaluations and summarizes critical data to be collected from the candidate SRRA, including ambient site considerations, background characteristics of the water supply, and SRRA wastewater characteristics.
Section 4—System Design Overview	Provides an overview of strategy for selection of an OWTS, including design approaches and procedures, ancillary requirements, and system layouts.
Section 5—Wastewater Collection	Provides design and selection procedures for an OWTS, including types of plumbing fixtures and connections, the drainage and sewer discharge system, lift station design, and grinder pump specifications.
Section 6—Primary Treatment	Provides procedures for selection of anaerobic baffled reactors, including process layout, design procedures, effluent management, and other considerations.
Section 7—Aerobic Treatment	Provides procedures for selection of the aerobic treatment system, including backwashing and packed bed filters; process layout, design procedures, and other considerations.
Section 8—Tertiary Filtration (Slow Sand Filter)	Provides procedures for selection of the slow sand filter, including process layout, design procedures, and other considerations.
Section 9—Color Removal (Ozonation)	Provides procedures for selection of the ozonation system, including process layout, design procedures, and other considerations.
Section 10—Disinfection (Chlorination)	Provides procedures for selection of the chlorination system, including process layout, design procedures, and other considerations.
Section 11—Recycled Water System Distribution	Provides descriptions of the infrastructure required for onsite recycling uses, such as toilet and urinal flushing, landscape irrigation, and storage.
Section 12—Residual Water Dispersal or Storage System	Provides residual characteristics and dispersal alternatives, including procedures for design of the dispersal or storage system; in most cases, the existing dispersal system can be used.
Section 13—Process Monitoring and Control	Provides a discussion of monitoring techniques, including manual and online methods, and control systems used for manual and remote operations for the management of onsite systems.
Section 14—References	Provides a list of documents cited in developing of this design manual.

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2.0 OVERVIEW OF ONSITE WASTEWATER TREATMENT SYSTEMS

Wastewater treatment processes are designed to achieve limits set for the effluent quality. Wastewater contains levels of suspended solids, biodegradable organics, nutrients, and pathogenic bacteria that must be removed from the waste stream prior to discharge or reuse. Constituents in wastewater can have negative effects on ecosystems and human health; therefore, OWTSs are designed to remove or reduce the concentration of these constituents to acceptable levels before the effluent is recycled or discharged back to the environment. The removal of pathogenic microorganisms is critical to preventing disease causing-organisms from entering drinking water supplies, water bodies, or plumbing fixtures that people may contact.

Section 2 discusses treatment process configurations, regulatory requirements applicable to OWTS, and an overview of OWTS operation, as outlined in Chart 2-1.

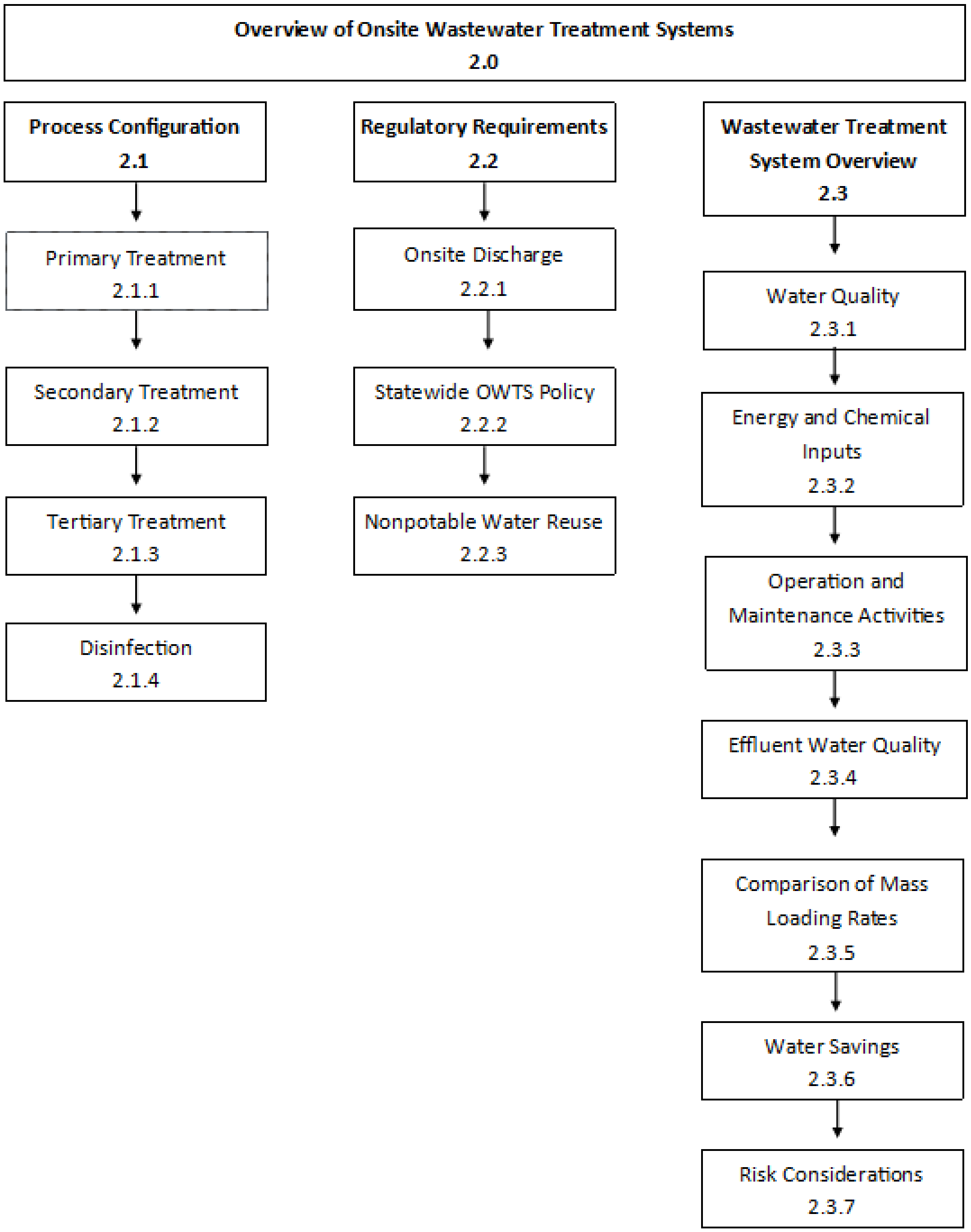

Chart 2-1
Overview of wastewater treatment process configurations

2.1 PROCESS CONFIGURATION

A conventional wastewater treatment is a combination of physical, chemical, and biological processes and operations to remove solids, organic matter, and, sometimes, nutrients from wastewater. At some sites, a pre-treatment step such as a screen or trash collection device may be required to control trash and prevent large debris from entering the OWTS and impacting subsequent treatment processes. Generally, the levels of wastewater treatment, in order of increasing treatment level are primary, secondary, and tertiary (or advanced). In some cases, treatment also includes disinfection to remove pathogens as the final treatment step. A sequence of treatment that utilizes primary treatment, secondary treatment, tertiary filtration, and disinfection is used to produce water that is free from human pathogens and safe for unrestricted nonpotable reuse. Each treatment stage is described below.

2.1.1 Primary Treatment

Primary treatment is designed to remove gross, suspended, and floating solids from raw wastewater, generally using gravity to allow solids to settle (sedimentation). Primary treatment involves basic processes, such as a septic or Imhoff tank, to remove suspended solid waste and to reduce its biochemical oxygen demand (BOD), by allowing microorganisms to break down organic material present. Sediments and other heavier-than-water components of the waste stream are allowed to settle out, while grease, oil, and other floatables (scum) float and are removed during this treatment stage. In a conventional system, raw wastewater comes out of primary treatment with its BOD reduced by 20 to 30 percent, and the total suspended solids are reduced by approximately 50 to 60 percent (Tchobanoglous et al., 2014). Some organic nitrogen, organic phosphorus, and heavy metals associated with solids are also removed during primary sedimentation, but colloidal and dissolved constituents are not affected. The effluent from primary settling units is referred to as primary effluent.

2.1.2 Secondary Treatment

Secondary (biological) treatment is used to further treat the effluent from primary treatment to remove the residual concentrations of organics and suspended solids. In most cases, secondary treatment removes biodegradable dissolved and colloidal organic matter using aerobic biological treatment processes. Secondary systems are designed to use microbes to consume organic matter, which is converted into carbon dioxide, water, and energy. These biological processes require a manageable biomass to feed on incoming waste loads. Secondary treatment technologies vary, and include basic activated sludge processes, variants of pond and constructed wetland systems, packed bed filters, and trickling filters; the biological process is generally followed by additional settling tanks to remove suspended solids. High-rate biological treatment processes combined with primary treatment generally remove about 85 percent of the remaining suspended solids and BOD.

2.1.3 Tertiary Treatment

In some cases, supplemental treatment processes are necessary to remove nitrogen, phosphorus, additional suspended solids, refractory organics, heavy metals, or dissolved solids. Similarly, effective disinfection is inhibited by the presence of suspended matter, as well as some colloidal and dissolved solids in the water, so these solids must be removed prior to the disinfection step.

Tertiary treatment is employed when suspended particulate wastewater constituents that cannot be removed by secondary treatment are removed by filtration through a granular media, synthetic fabric, or membrane. If needed, chemical oxidation is used to enhance the removal of colloidal and dissolved constituents.

2.1.4 Disinfection

Disinfection is the final step in the wastewater treatment process. Disinfection, typically with chlorine, ultraviolet light, or ozone, individually or in combination, is the final step before reuse or discharge of the water. Effluent is dosed at a controlled level of disinfectant to remove or inactivate any remaining pathogens. When using disinfectants that retain a residual effect, instrumentation monitoring the residual concentration can help determine the dosage required for disinfection. Dosage depends upon the strength of the wastewater and other factors. Virus removal of disinfectants may depend upon total dissolved solids (TDS), pH, contact time, organic content, and effluent temperature.

2.2 REGULATORY REQUIREMENTS

The regulations most relevant to SRRA OWTSs are standards for discharge to groundwater and requirements for water reuse.

2.2.1 Onsite Discharge

As of October 2015, wastewater systems serving the SRRAs are under the jurisdiction of the RWQCBs through the Porter-Cologne Act (Clean Water Act) and may be regulated under the applicable regional basin plan. In some cases, basin plan requirements may establish the basis of the siting, design, and permitting of OWTSs for the region. Systems handling flows or waste strength (concentration) similar to those at the SRRAs are typically regulated by the RWQCB under waste discharge requirements (WDRs) that permit the discharge to the waters of the State; however, in some cases, the county may be delegated responsibility for regulation. These WDRs are issued either through the General Order (discussed below) or as individual WDRs with site-specific effluent discharge limits. Individual WDR limits are based on a multitude of parameters, including daily flow, waste strength, site conditions, and threat to water quality.

In some cases, the RWQCB enrolls projects under a broader SWRCB Order WQ 2014-0153-DWQ – General Waste Discharge Requirements For Small Domestic Wastewater Treatment Systems (http://www.waterboards.ca.gov/board_decisions/adopted_orders/water_quality/2014/wqo2014_0153_dwq.pdf), referred to as the General Order. For systems to be covered under the General Order, specific conditions must be met for the daily flow, waste strength, and site conditions.

2.2.2 Statewide OWTS Policy

The SWRCB adopted the OWTS Policy to establish a statewide, risk-based, tiered approach for the regulation and management of OWTS installations and replacements, and to set the level of performance and protection expected from OWTSs with onsite disposal or dispersal of effluent.

The tiers are summarized as follows:

- Tier 0: Properly functioning existing OWTSs
- Tier 1: Low-risk new or replacement OWTSs
- Tier 2: Local agency management plan for new or replacement OWTSs
- Tier 3: Existing, new, and replacement OWTSs that are near impaired water bodies
- Tier 4: OWTSs that require corrective action

The OWTS Policy went into effect on May 13, 2013, and set standards for OWTSs "that are constructed, replaced, subject to a major repair, or pool waste or discharge it to the surface of the ground, and that have affected or will affect groundwater or surface water to a degree that makes it unfit for drinking water or other uses, or causes a health or other public nuisance condition (SWRCB, 2012)".

The OWTS Policy covers systems with projected flows of less than 10,000 gallons per day (gal/d), and have a 30-day (d) average waste strength of less than 300 milligrams per liter (mg/L) of BOD and less than 330 mg/L of total suspended solids (TSS). While most SRRA facilities with modern low-flow plumbing fixtures have flows less than 10,000 gal/d, the use of water-conserving plumbing fixtures results in a concentrated waste stream that exceeds the 300 mg/L limit for BOD and TSS. Therefore, it is expected that many SRRA wastewater systems will be regulated using WDR by the RWQCB, as discussed in Sec. 2.2.1, because of the characterization of the wastewater as a high-strength waste stream.

The OWTS Policy also includes:

- Minimum operating requirements for OWTS, which may include siting, construction, and performance;
- Requirements for OWTS near certain waters listed as impaired under Section (§) 303(d) of the Clean Water Act;
- Authorization of the local agency to implement the requirements;
- Corrective action requirements;
- Minimum monitoring requirements;
- Exemption criteria;
- Requirements for determining when an existing OWTS is subject to major repair; and
- A conditional waiver of WDRs.

The RWQCBs were required to incorporate the standards established in the OWTS Policy, or standards that are more protective of the environment and public health, into their water quality control plans by May 14, 2014. Implementation of the OWTS Policy is overseen by the SWRCB and the RWQCBs. Local agencies (e.g., county and city departments and independent districts) have the opportunity to implement local agency management programs (LAMPs), if they are approved by the applicable RWQCB.

The OWTS Policy also addresses OWTSs that are near impaired water bodies pursuant to §303(d) of the Clean Water Act. In general, if the OWTS is within 600 feet (ft) of a water body listed as impaired because of nitrogen or pathogens and there is no total maximum daily load (TMDL) requirement, OWTSs must meet the applicable requirements of Tier 3. Requirements that apply to a specific SRRA will be determined on a case-by-case basis or as part of a separate statewide study. Typically, Tier 3 requirements are:

- Coverage under an Advanced Protection Management Plan (APMP);
- Increased vertical and horizontal setbacks;
- Supplemental treatment meeting the performance standards of 50 percent reduction in nitrogen, based on a 30-day average influent and effluent; and
- If impaired for pathogens, achievement of an effluent that does not exceed 30 mg/L of TSS or fecal coliform numbers of 200 most probable number (MPN)/100 milliliters (mL).

The APMP is defined by the TMDL or by an approved local area management plan (Tier 2). Attachment 2 of the OWTS Policy lists impaired water bodies that are near potential OWTSs that are determined to be a contributing source of pathogens and nitrogen.

For impaired water bodies that have an adopted TMDL addressing the impairment, but no waste load allocation is assigned to the OWTS, no further action is required unless the TMDL is modified to include actions for the OWTS. Existing, new, and replacement OWTSs that are near impaired water bodies and are covered by a basin plan prohibition must also comply with the terms of the prohibition, as provided in Sec. 2.1 of the OWTS Policy.

2.2.3 Nonpotable Water Reuse

The regulation of water reuse in California falls under the California Code of Regulations (CCR) Title 22 Social Security, Division 4 Environmental Health, Chapter 3 Water Recycling Criteria (Title 22). Title 22 is the principal regulation for water reclamation and has the most stringent water recycling criteria in the United States (U.S.) (Asano et al., 2007).

Potential reuse applications of disinfected tertiary effluent at SRRAs include toilet and urinal flushing, and spray irrigation (e.g., sprinkler systems). Disinfected tertiary treatment is not required for subsurface irrigation (Title 22 §60301.230, §60301.320,

§60307, and §60321). The potential practice of water reuse at SRRA facilities would be regulated currently by the RWQCB and the SWRCB Division of Drinking (DDW) Title 22 recycle program. Per the current SWRCB DDW Title 22 requirements, while operating a wastewater reuse treatment system, the treated disinfected tertiary water (OWTS effluent) is required to be monitored daily for total coliform bacteria.

Note that §60320.5 allows for the demonstration of other treatment methods for compliance. For example, it has suggested that daily bacteria sampling could be waived by putting in two storage tanks that can each hold one week's worth of water supply. Weekly sampling from each batch tank could then be considered adequate. This alternative option would increase the upfront construction costs, but would decrease the maintenance sampling costs. Online sensors that can detect water quality parameters that are correlated with disinfection performance have also been proposed as an alternative to daily bacteria sampling. Onsite sensors will increase the capital cost but reduce the operations and maintenance cost.

The sections of Title 22 as they apply to semi closed-loop, onsite wastewater reuse treatment systems, such as those at the Dunnigan SRRA pilot study facility, are summarized in Table 2-1 and Table 2-2. For nonpotable reclaimed water, including that used for toilet and urinal flushing, the water must be of "disinfected tertiary" quality, as defined by the disinfection process and coliform bacteria concentration. Disinfected tertiary processes are categorized as either chlorine-treated or other. All disinfection systems other than chlorine disinfection must demonstrate disinfection capacity through validation testing (challenge testing). Note that chlorination of water with high organic matter may cause the formation of trihalomethanes (THM), which are a concern in drinking water and surface water (i.e., river and lake) discharge, neither of which is intended for the water treated at SRRAs.

Table 2-1
Summary of CCR Title 22 filtration requirements

Filtration requirements	Parameter	Coagulated and filtered through filter bed	Filtered through filter bed without coagulation[1]	Microfiltration; reverse osmosis membrane
Design requirements	—	—	Automated chemical addition or diversion if filter influent >5 NTU for 15+ min	—
Allowable concentrations	Turbidity 24-h avg	2 NTU	2 NTU	—
	5% of 24-h single turbidity values	5 NTU	5 NTU (15-min maximum)	0.2 NTU
	Turbidity not to exceed	10 NTU	10 NTU	0.5 NTU
Continuous monitoring requirements[2]	24-h avg		4-h intervals	
	5% of 24-h single values		1.2-h intervals	
	Reporting to regulatory agency		Quarterly	

1. Treatment process is allowed only for specified reuse purposes (e.g., toilet flushing).

2. Continuous monitoring is required after the filter when filtration is coagulated and filtered through a filter bed, before and after the filter when filtered through a filter bed without coagulation, and after filter with microfiltration.

% = percent; avg = average; h = hour; min = minute; NTU = nephelometric turbidity unit

Table 2-2
Summary of CCR Title 22 disinfection requirements

Disinfection requirements	Parameter	Chlorine disinfection	Other disinfection
Design requirements	—	Filtration required	Demonstrate a 5-log (99.999%) inactivation of F-specific bacteriophage MS-2
	—	C_RT ≥450 mg-min/L	
	—	Modal C_RT ≥90 min (peak dry weather flow)	
Allowable concentrations	Total coliform 7-d median	2.2 MPN/100 mL	
	Total coliform 30-d single value	23 MPN/100 mL	
	Total coliform single value	240 MPN/100 mL	
Monitoring requirements	Total coliform	Daily sampling	
	Residual	Continuous	—

2.3 WASTEWATER TREATMENT SYSTEM OVERVIEW

The water recycling regulations in the CCR were created with centralized facilities in mind and, as a result, may impede the implementation of small, decentralized wastewater treatment and reuse systems. The absence of onsite wastewater reuse treatment systems in the California regulations is recognized in the California Water Code §13553.1, which states that “there is a need for a pilot program to demonstrate that conversion to the use of recycled water in residential buildings for toilet and urinal flushing does not pose a threat to public health and safety” (Office of Administrative Law, 2012). To date, a number of pilot programs and systems exist that demonstrate this safety (e.g., at the San Francisco Public Utilities Commission Headquarters, 525 Golden Gate Avenue in San Francisco); however, the results have yet to be reviewed and incorporated into statewide regulatory criteria.

The pilot OWTS evaluated at the Dunnigan SRRA pilot facility serves as a demonstration and aims to guide the formation of a regulatory structure more suited to small and decentralized water reuse systems. The water recycling system consists of the following unit operations: (1) anaerobic baffled reactor, (2) backwashing filters, (3) slow sand filtration, (4) ozonation, and (5) chlorination. A schematic of the process is shown on Fig. 2-1. The process configuration was developed over a two-year pilot testing period using both commercially available and custom-engineered processes. The focus of the pilot testing was to optimize performance and reliability, such that the treatment process is automated, operated, and monitored remotely, and requires minimal onsite operations.

The processes shown on Fig. 2-1 were selected for their overall reliability, simplified maintenance, and ability to operate for extended periods of time (from months to years) without any service needs. Through a pilot test program that involved testing of alternative treatment trains and various loading rates, a final process configuration was developed that provides the highest level of reliability.

Important features of the system operation are:

- Operation at constant flowrate and loading (eliminating process response to variable loading patterns),
- Ability to shut down the process for several days, e.g., power outages, and restart it without impacts on process performance,
- Redundant treatment systems, and
- Connection for a supervisory control and data acquisition (SCADA) system for online monitoring and control.

2.3.1 Water Quality

The operational characteristics and effluent quality from the process are compared with typical recycled water quality specifications for unrestricted reuse in Table 2-3.

The feasibility of onsite water reuse systems would be impacted negatively by requirements developed for municipal water recycling, such as daily coliform measurements. As described in Sec. 2.1, it is proposed that a high level of treatment, achieved through process redundancy and continuous online monitoring, will be used to demonstrate process performance in lieu of conventional monitoring requirements.

Another option that was discussed is the storage of a volume of treated water, e.g., 30,000 gal, while the bacterial concentration is determined and prior to use as a recycled water supply. There are several reasons that the option of water storage is not discussed further: (1) chlorine residual will decay during storage due to natural dissipation and additional chlorine would need to be added to control bacterial growth during the storage period, (2) weekly bacterial measurements would still be required, increasing the operation and maintenance costs, and (3) the capital cost and physical footprint of treatment systems would be increased significantly by the need for multiple large storage tanks.

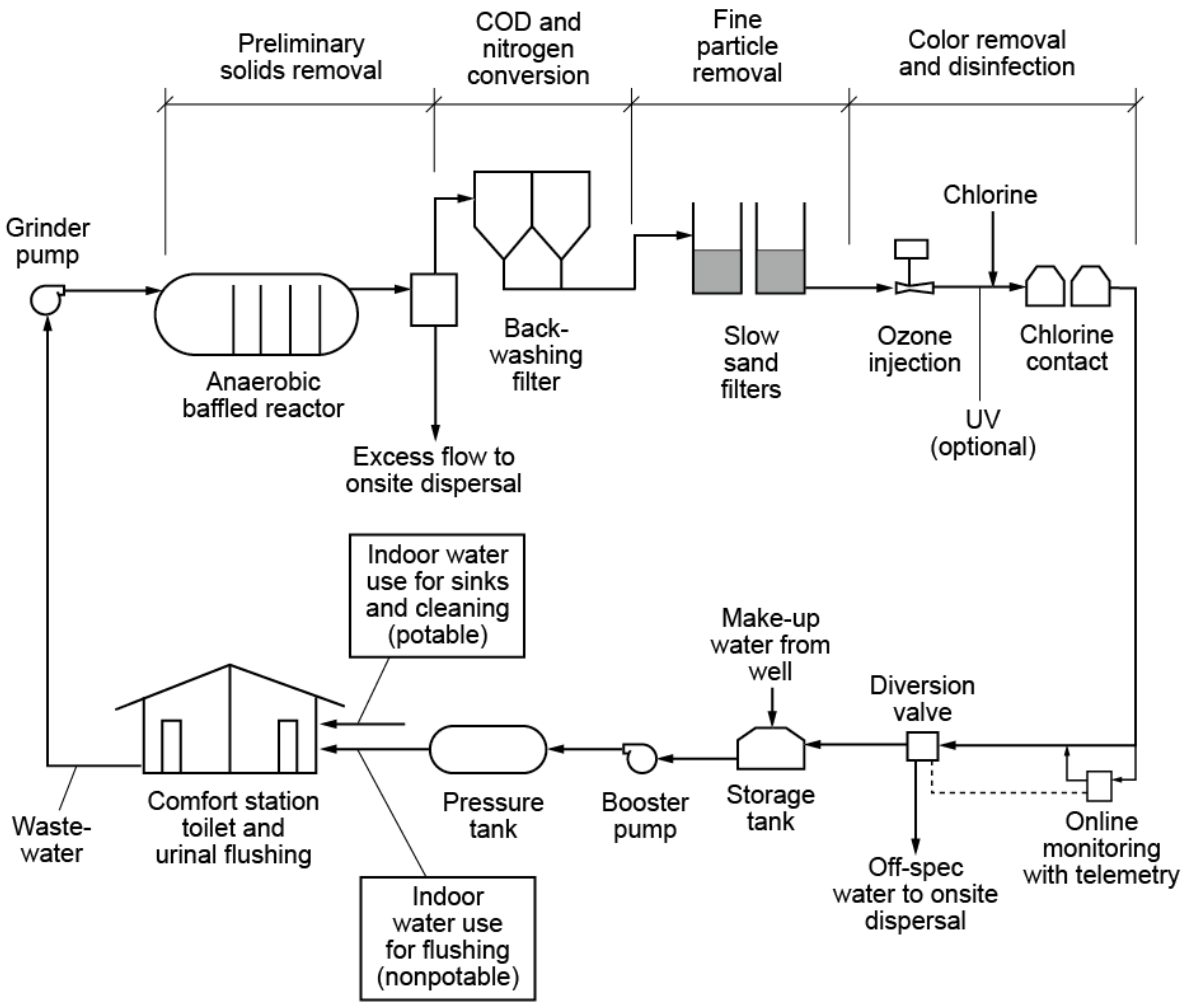

Figure 2-1
Generic water reuse treatment process configuration summary

2.3.2 Energy and Chemical Inputs

The production of recycled water uses electrical energy for a number of process elements, including pumps, controls, and ozone generation. The typical power consumption is summarized in Table 2-4.

The chemical inputs include chlorine and alkalinity. Sodium hypochlorite is applied to maintain a combined chlorine residual around 5 mg/L. Sodium bicarbonate is used to adjust the final product water alkalinity to 50 mg/L to protect plumbing from corrosion during water reuse.

Table 2-3
Comparison of CCR Title 22 with the proposed onsite water recycling system

Parameter	CCR Title 22 requirements for toilet and urinal flushing	Proposed process
	Filtration process	
Turbidity, NTU		
24-h avg	2	< 2
Not to exceed	10	10
Monitoring	Continuous (before and after filter)	Continuous (after filter[a])
Design	Automated chemical addition or diversion if filter influent >5 NTU for 15+ min	Automated diversion if turbidity exceeds 2 NTU
	Disinfection process	
Chlorine	Filtration required CT ≥ 450 mg-min/L Modal contact ≥ 90 min	Filtered CT > 1500 mg-min/L Modal contact > 300 min
Other processes	Demonstrate a 5-log (99.999%) inactivation of MS-2	Ozone achieves > 12-log reduction in MS2
Total Coliform, MPN/100 mL		
7-d median	2.2	< 2.2
30-d single value	23	< 23
Single value	240	< 240
Coliform monitoring	Daily sampling	Not required if turbidity and color or chlorine residual are within specified range
Chlorine residual	Continuous	Continuous
	Process integrity	
Color / UVA	Not required	Continuous
pH	Not required	Continuous
ORP	Not required	Continuous

ORP = oxidation-reduction potential; UVA = UV absorption

a. Monitoring of turbidity before filter is optional

2.3.3 Operation and Maintenance Activities

As described above, the selected treatment train utilizes technologies that were developed for long-term deployment with limited operation and maintenance requirements. Based on the results of extended pilot testing, the maintenance schedule shown in Table 2-5 has been established. As summarized in Table 2-5, the routine operation and maintenance activities will take place on a monthly basis. The monthly basis is justified because of the robustness of the system. Additional maintenance activities per process are described in the Operation, Maintenance, and Troubleshooting sections for each respective treatment unit, and summarized in Appendix A.

In the event of a process upset or alarm, a number of redundant and backup systems are available to maintain public safety. The system operator will be automatically notified of the alarm. In the event that the alarm is associated with final effluent quality monitoring, an automated valve will discharge the off-spec recycled water to the onsite treatment and dispersal system. Groundwater from the site will be supplied automatically until the alarm condition is cleared or whenever supplemental water is needed. The recycled water system is designed to be supplementary to a fully functional water and wastewater system capable of managing the entire facility demand.

Table 2-4
Summary of estimated energy requirements to produce 2500 gal/d of recycled water based on Dunnigan SRRA pilot study

Process component	Power, kWh/d[a]	Estimated electricity cost annually, $[b]
Lift station pumps	1.3	48.45
Backwashing filter	4.0	146
Slow sand filter	0	0
Ozonation	22	803
Chlorination	0	0
Booster pumps	2.0	73
Process total	29	1059
Normalized to 1000 gal	11.7	427

a. kWh/d = kilowatt hour per day
b. Estimated costs based on assumed unit cost for electricity of $0.10/kWh

Table 2-5
Summary of proposed routine operation and maintenance activities associated with water recycle systems at SRRAs

Activity	Frequency	Time, h	Cost[a], $	Annual cost, $
Process checks	Monthly	4 - 6	600	7200
Calibrate chlorine analyzer	Monthly	0.25	25	300
Calibrate turbidity analyzer	Monthly	0.25	25	300
Prepare chlorine solution	Monthly	0.25	25	300
Field measurements	Monthly	2	200	2400
Site cleanup	Monthly	2	200	2400
Water quality monitoring system flush	Quarterly	2	200	800
Scrape sand filter	Quarterly	4	400	1600
Replace gaskets on oxygen generator	Annually	2	200	200
Rebuild pH and chlorine electrodes	Annually	2	200	200
Remove solids from process tanks	Annually	4	400	400
Replace lead tubes in metering pumps	Annually	1	100	100
Travel time, annual[b]		48	4800	4800
Total[a]				19000

a. Based on $100/h
b. Based on 1-h travel time each way

2.3.4 Effluent Water Quality

The potable water supply at most SRRAs is derived typically from groundwater or local spring water. In most cases, the potable supply is used once, treated to some extent, and then dispersed in upper soil horizons. Water supply at SRRA facilities is used for hand washing, drinking, toilet and urinal flushing, cleaning, and irrigation. The typical raw effluent characteristics from SRRAs are summarized in Table 2-6 (see column Single-use effluent). Based on the readings from water meters, toilet and urinal flushing accounts for about 90 percent of indoor water use (handwashing accounts for 5 to 10 percent of the indoor water use); therefore, the maximum amount of nonpotable recycled water is about 90 percent. Residual water will be discharged to the existing onsite treatment system (see column Process effluent in Table 2-6). The recycled water and process effluent will have an elevated salt, (i.e., TDS), concentration from the high level of water efficiency, as shown in Table 2-6. Constituents that accumulate in the recycled water (e.g., dissolved solids) will therefore be discharged at a higher concentration as a consequence of the high water use efficiency.

2.3.5 Comparison of Mass Loading Rates

While constituents that are not amenable to treatment, such as sodium and chlorine, will be discharged at a higher concentration compared with the single-use scenario, the mass loading of most constituents to the environment will be reduced significantly. As summarized in Table 2-7, the total mass loading to the onsite dispersal system will be reduced by over 90 percent for most constituents. In addition, the flow of wastewater to the onsite dispersal system will be reduced by 90 percent.

Table 2-6
Summary of expected water quality values for recycled water and process effluent

Constituent or parameter	Unit	Single-use effluent (no recycle)	Recycle water (assumes 90 percent indoor water recycled)	Process effluent (excess/residual flow)
Flow	gal/d	2750	2500	250
BOD	mg/L	550	10	50
COD	mg/L	1250	25	200
TSS	mg/L	800	<1	50
TDS	mg/L	1200	3000 - 4000	3000 – 4000
TKN	mg N/L	225	10	250
TON	mg N/L	-	200	-
pH	-	8.3	7.5	7.8
Total coliform	MPN/100 mL	108	0	105

COD = chemical oxygen demand; mg N/L = milligrams of nitrogen per liter; TKN = total Kjeldahl nitrogen; TON = total oxidized nitrogen, i.e., total nitrite and nitrate

Table 2-7
Summary of expected mass loading values for recycled water and residual process effluent for typical SRRA facility

	Mass loading, kilograms per day (kg/d)		
Constituent or parameter	Single-use effluent	Recycle water (90 percent recycle)	Effluent to onsite dispersal system
BOD	5.7	0.10	0.05
COD	13.1	0.24	0.19
TSS	8.4	0.0	0.05
TDS	12.5	33.3	3.3
TKN	2.4	0.48	0.24
TON	-	1.9	-

2.3.6 Water Savings

It is estimated that the proposed water recycle system could produce approximately 2700 gal of recycled water per day and reduce the use of potable water by more than 1 million gallons per year (Mgal/y) for each SRRA installation when used for toilet and urinal flushing. If the water is used for meeting irrigation demands, the overall amount of water savings will be reduced because of the seasonal nature of irrigation systems. At facilities with older fixtures, such as Dunnigan-NB SRRA, the combination of low-flow fixtures, waterless urinals, and nonpotable recycled water toilet flushing would reduce potable water use by about 2-Mgal/y, or approximately a 95 percent reduction in potable water use.

2.3.7 Risk Considerations

The primary routes of exposure to water used for toilet flushing are (1) direct contact if someone makes contact with the water (e.g., retrieves a dropped phone from the bowl), (2) skin contact from splashing during defecation, and (3) breathing of aerosols created when a toilet is flushed. The risk associated with these exposure routes when using recycled water is reduced through the application of robust treatment systems. The recycled water system has a number of features that make process failure and risk to the public unlikely:

- Full back-up water and wastewater systems for complete process redundancy;

- Continuous remote monitoring of process water quality;
- SCADA for remote process control;
- Automatic diversion for off-spec water;
- Redundant disinfection;
- Technical support and on-call operations staff; and
- Design based on results from long-term pilot testing.

See Sec. 13 for a more detailed discussion of the aforementioned risk consideration features, and process monitoring and control requirements.

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3.0 SRRA WASTEWATER CHARACTERISTICS

The design of onsite wastewater treatment and recycling systems requires accurate information of parameters such as characteristics of the site and SRRA facility, background data on the potable water supply, and facility usage rates for estimating wastewater constituent loading. A summary of preliminary data development for design of a water reuse system is presented in this section.

Section 3 comprises information on wastewater characteristics needed to estimate indoor water use and wastewater generation rate and compute design loading parameters, as outlined in Chart 3-1.

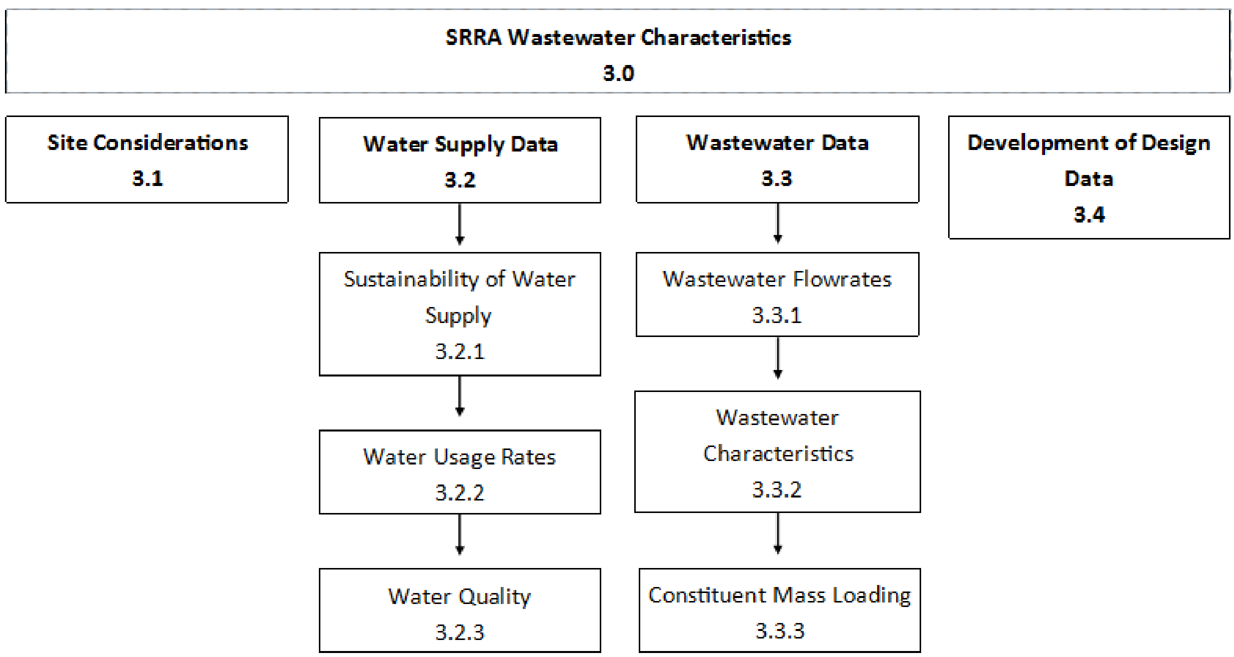


Chart 3-1
Preliminary system design steps

3.1 SITE CONSIDERATIONS

Each SRRA site is unique, not only in terms of loading and usage factors, but also in weather, topography, and other features that impact the siting and design of the water systems. Factors that need to be investigated during the design process are presented in Table 3-1.

Table 3-1
Summary of site considerations for the development of SRRA water systems

Item	Considerations
Water supply	Potable water supply can have some impact on water recycling operations because of mineral content; however, at high levels of water recycling, the chemistry of the potable supply is minimized.
Local weather	Local climate needs to be investigated to determine the temperature fluctuations, as biological treatment processes can be impacted by water temperature. In some cold climates, the water temperature in aboveground water tanks can drop significantly and possibly freeze if tank heaters are not used.
Site and space availability	Sufficient space is needed for process equipment, which includes lift stations, treatment tanks, and recycled water storage and distribution. In addition, space is required for vehicle access to the site for operation and maintenance. The typical footprint is 36 ft by 80 ft. At some sites, it may be necessary to break the process components up spatially to meet challenging space constraints. The right-of-way limit should also be investigated for system placement and possible additional space, if required.
Plumbing systems	Dual plumbed water systems are used to provide potable water for sinks and fountains, and nonpotable water for toilet flushing and irrigation. An inspection of the plumbing system will be needed to determine whether potable and nonpotable supply piping are pre-existing. In addition, the plumbing system should be inspected for signs of corrosion, modifications, and accessibility.
Storm flood history	At SRRA facilities with a history of flooding, the placement of process tanks and control systems will require special attention. Process equipment must be water tight and properly anchored under all conditions because flooding has the potential to wash out the process and/or cause surface discharge. Control systems must remain operational at all times when the facility is in use to prevent overflows.
Groundwater elevation	At sites with high groundwater or construction during the winter season, it is important that tanks are properly anchored according to manufacturer specifications. Tanks that are empty or partially empty can be pushed from the ground because of buoyancy as groundwater rises or the excavation fills with water if not adequately anchored.
Utility investigation	Both aboveground and underground utilities should be considered in the planning process. In addition to underground utilities such as electrical, phone, and water supply, abandoned wastewater system components, gas mains, fuel lines, and data cables will need to be considered. Access to all types of utilities should be maintained.
Power requirements	The power required to operate a recycled water system will be greater than for conventional septic systems. An investigation of the power supply is recommended to determine whether an upgraded service connection will be required.
Site history	Some sites are located in areas with vandalism concerns. Approaches to mitigating vandalism have ranged from securing or removing items prone to vandalism to installing video monitoring and surveillance equipment (monitored by maintenance personnel or by a contractor) to deter vandalism. The video monitoring systems and related signage appear to be more effective in preventing theft and vandalism.

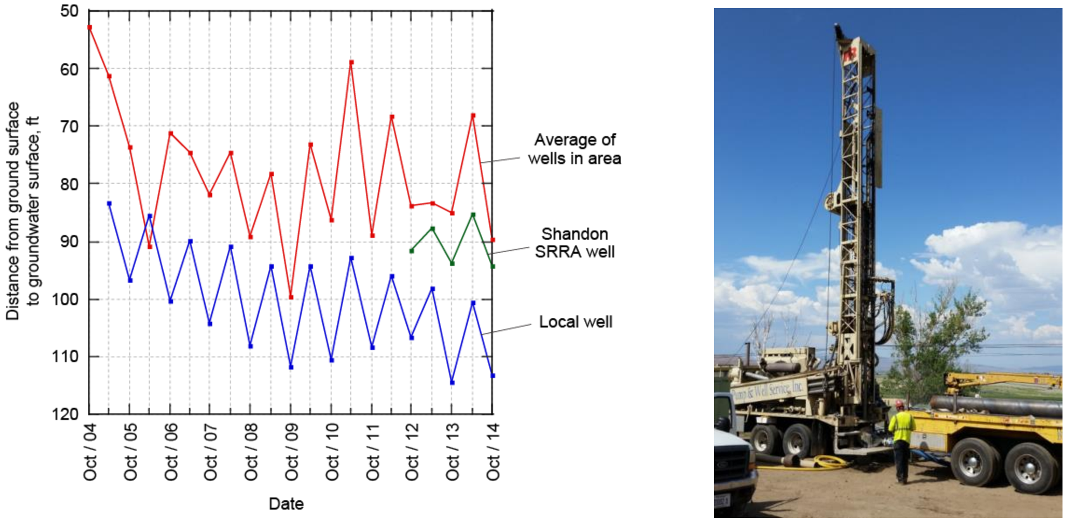
3.2 WATER SUPPLY DATA

The availability of water with suitable quality is essential for the proper operation of SRRA facilities. Water is used at SRRA facilities for the transport of human waste, handwashing, and drinking. However, most indoor water is used for flushing toilets and urinals. The topics covered in this section include existing and projected water supply, water usage rates, and water quality.

3.2.1 Sustainability of Water Supply

The potable water supply used at SRRA facilities can be derived from several sources, including groundwater, streams, and springs. In most cases, access to a municipal supply is not an option because of the remote nature of SRRA facilities. It is anticipated that in the future rainwater could be used to supplement potable water supplies if other conventional sources become less reliable. The long-term sustainability of a given water supply depends on a number of factors, including cost of procurement, storage in aquifers, production from springs, and water quality. In many locations, groundwater levels have been declining due to drought and long-term over withdrawal. For example, historical declines in groundwater levels near the Shandon SRRA are shown on Fig. 3-1a. Other SRRAs have contaminated aquifers and need to develop deeper wells, as shown on Fig. 3-1b.

As a specific example of water supply concerns, the potable and nonpotable water supply for the Gaviota SRRA is derived from a spring source that flows at approximately 15 to 25 gallons per minute (gal/min) (California State Parks, 2013). The single spring source must be split among Gaviota SRRA, Gaviota State Park, recreation uses, and residential uses. The reliability of the water supply has been an ongoing issue at the Gaviota SRRA and at Gaviota State Park. Whenever there is an extended breakdown in the system, the rest areas must be closed as there is only enough water stored to cover operations for a short period of time. The water situation at the Gaviota SRRA, while unique because of its split uses, is an example where reuse of onsite treated effluent would result in better service to the public and a significant reduction in potable water use, and would take pressure off the potable water supply. Treated effluent can be used for both irrigation and toilet and urinal flushing.



(a) (b)

Figure 3-1
Groundwater concerns: (a) historical groundwater elevation measurements from San Luis Obispo (SLO) County water resources unit for aquifer used by the Shandon SRRA, and (b) drilling of deep water well at the Honey Lake SRRA because of nitrate contamination of the upper aquifer

3.2.2 Water Usage Rates

Water is used at SRRA facilities for a number of activities, including landscape irrigation, toilet and urinal flushing, handwashing, drinking, and indoor/outdoor cleaning. In addition, jug fillers are provided for the public to access the potable water supply. Water is discharged to the wastewater collection system through indoor water use activities such as fixture flushing, washing basins, water fountains, and floor drains. The amount of water used for each of these activities depends on the number of people using the facility, fixture flowrate, mechanism for actuation, maintenance, and other factors.

An example of the impact of plumbing fixture selection at the Dunnigan-NB SRRA is shown on Fig. 3-2. In August 2015, the toilets at the Dunnigan-NB SRRA were changed from 3.5 gallons per flush (gal/flush) to 1.28 gal/flush. The urinals were also changed from 1 gal/flush to waterless type urinals. After the SRRA facility opened with the new toilets, the water use was reduced from about 5000 to 2500 gal/d (comparing August 2014 and 2015). Even though multiple flushes have been observed with the new toilets because of problems flushing the paper seat covers, the water use has been reduced by more than 50 percent.

Probability distributions from the Dunnigan-NB SRRA are shown on Fig. 3-3. Several observations can be made about the data shown on Fig. 3-3. Most importantly, all distributions shown on Fig. 3-3 have a log-normal distribution, as demonstrated by the linear distribution when plotted on a logarithmic ordinate axis; i.e., the logs of the values are normally distributed. Most natural phenomena have been found to have a log-normal distribution. Because the SRRA usage data also have a log normal distribution, these data can be modeled on the basis of an analysis of the mean and peak values.

The geometric mean (50 percent value) of the traffic data shown on Fig. 3-3a for the summer season, year, and winter season is 1,450, 1,280, and 1,140 vehicles per day (veh/d), respectively. Based on this analysis, the seasonal variation can be estimated using a factor of 1.12. The peaking factor (99 percent value/50 percent value) for the annual data is 1.6, based on a peak value of 2,050.

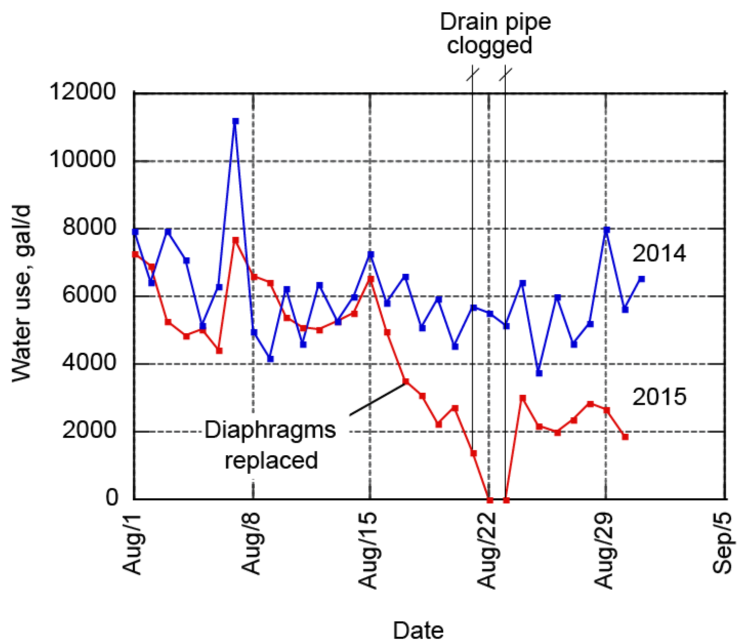


Figure 3-2
Comparison of flow data at the Dunnigan-NB SRRA before and after replacement of the 3.5-gal/flush toilets and 1.0-gal/flush urinals with 1.28-gal/flush toilets and waterless urinals (on August 17, 2015)

The geometric mean of the people count data shown on Fig. 3-3b for the summer season, year, and winter season is 1710, 1475, and 1270 veh/d, respectively. Based on this analysis, the seasonal variation can be estimated using a factor of 1.16. The peaking factor for the annual data is 2, based on a peak value of 2920.

The geometric mean of the water use data shown on Fig. 3-3c for the summer season, year, and winter season is 5130, 4420, and 3810 veh/d, respectively. Based on this analysis, the seasonal variation can be estimated using a factor of 1.16. The peaking factor for the annual data is 2, based on a peak value of 8850.

The seasonal use and peaking factors, summarized in Table 3-2, can be used for predicting the distributions at other facilities. A procedure for estimating the SRRA indoor water use flowrate, and by extension the rate of wastewater generation, is presented below. Use of these relationships is shown in Ex. 1.

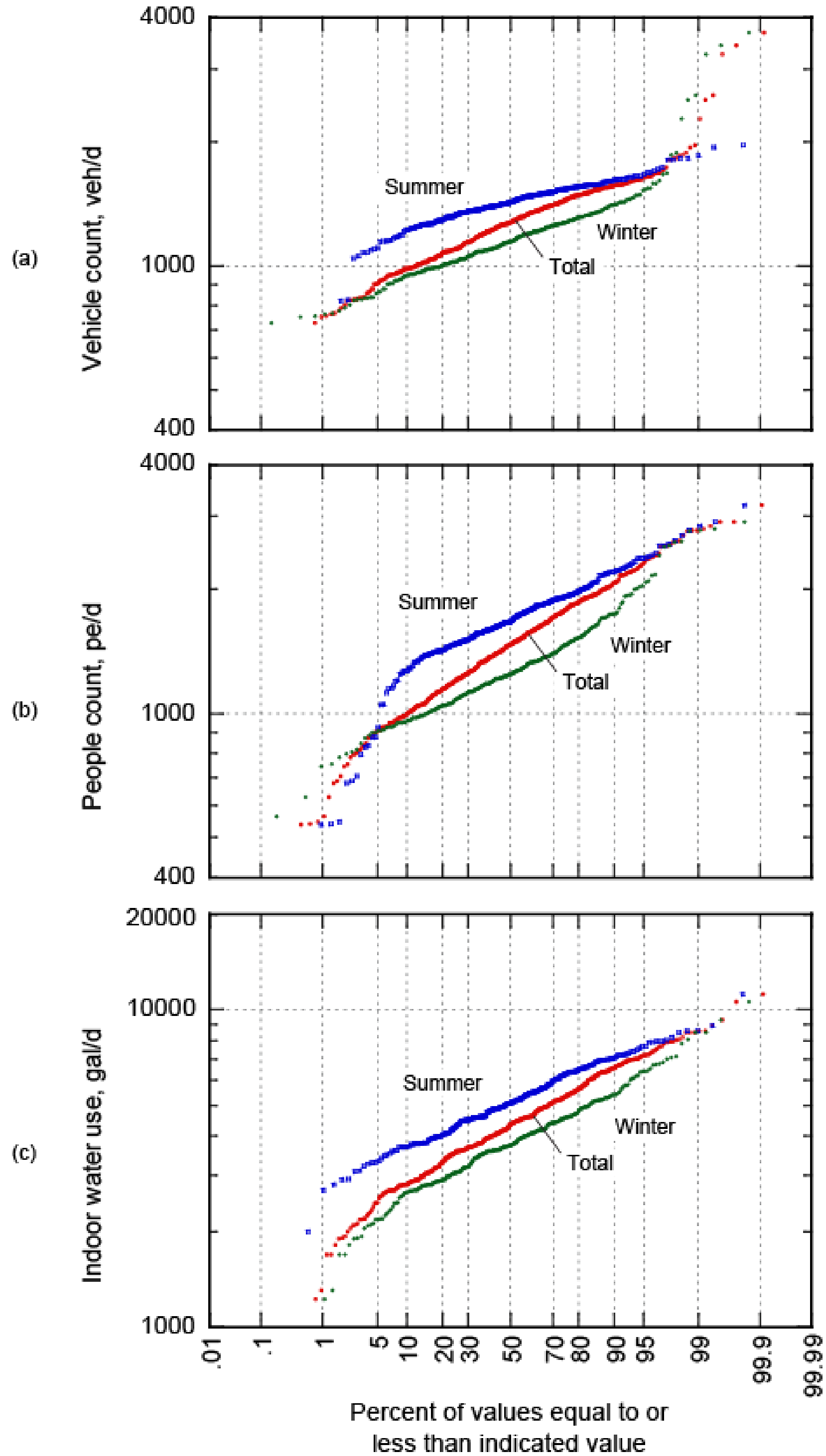


Figure 3-3

Probability distribution of wastewater process data at the Dunnigan-NB SRRA prior to August 2015 plumbing upgrades: (a) vehicle counts, (b) people counts, and (c) indoor water use. The probability axis is used to relate the measured values to the frequency of occurrence; for example, the values corresponding to 99 and 50 percent on the probability axis are taken typically to be the peak and mean values for a data set.

Table 3-2
Summary of parameters used for modeling water use at SRRAs

Parameter	Unit	Typical value	Notes
Peaking factor, daily basis	-	2.0 (flow) 2.0 (people) 1.6 (traffic)	The peaking factor is used to estimate the maximum expected usage rates for the SRRA and is defined at the value that is exceeded with a frequency of 1 percent divided by the geometric mean. Based on average annual data.
Peaking factor, hourly basis	-	2.4	The peak use is spread out over about a 10-h period. The hourly flowrate and people count during this 10-h period can be estimated as the product of the hourly average and the hourly peaking factor.
Use escalation, factor	-	1.3	Typical multiplier used to project increased usage based on estimated OWTS lifespan of 30 years in the future
Seasonal variation, factor	-	1.16 (flow) 1.16 (people) 1.12 (traffic)	Multiplier to estimate seasonal change in the geometric mean value in summer (March through August) and winter (September through February)
Vehicle counts	veh/d	-	Reference available ramp count data
People per vehicle	pe/veh	1.3 (I-5) 3.3 (US-101)	The number of people per vehicle is dependent on the primary type of traffic, e.g., truck traffic on I-5 and commuter and tourist traffic on US-101
Men, fraction Women, fraction	- -	0.55 0.45	It is common to have slightly more men visitors than women
Men using urinals, fraction	-	0.80	Most men use the urinal when available
Urine volume	gal/pe	0.055	Expected volume of urine per person
Factor for valve fouling	-	1.0 to 1.5	Flush valve diaphragms have been found to deteriorate and foul over time. Applies to both toilets and water flush urinals. A factor of 1 is used if the diaphragms are replaced on a regular basis. A larger factor should be used if regular O&M is not anticipated.
Toilets (gal/flush), factor 3.6 1.6 1.28	flush/pe	 1.1 1.25 1.5	A flush factor is applied to toilets account for excess water use when multiple flushes are required to clear the bowl or hyperactive flush sensors
Handwashing	gal/pe	0.125	Expected amount of water used for handwashing
Cleaning	gal/d	50	Expected amount of water used for daily cleaning activities in the comfort station

flush/pe = flushes per person; gal/pe = gallons per person; O&M = operations and maintenance

The design procedure for estimation of indoor water use and wastewater generation rate is summarized on Chart 3-2, and demonstrated in Ex. 1.

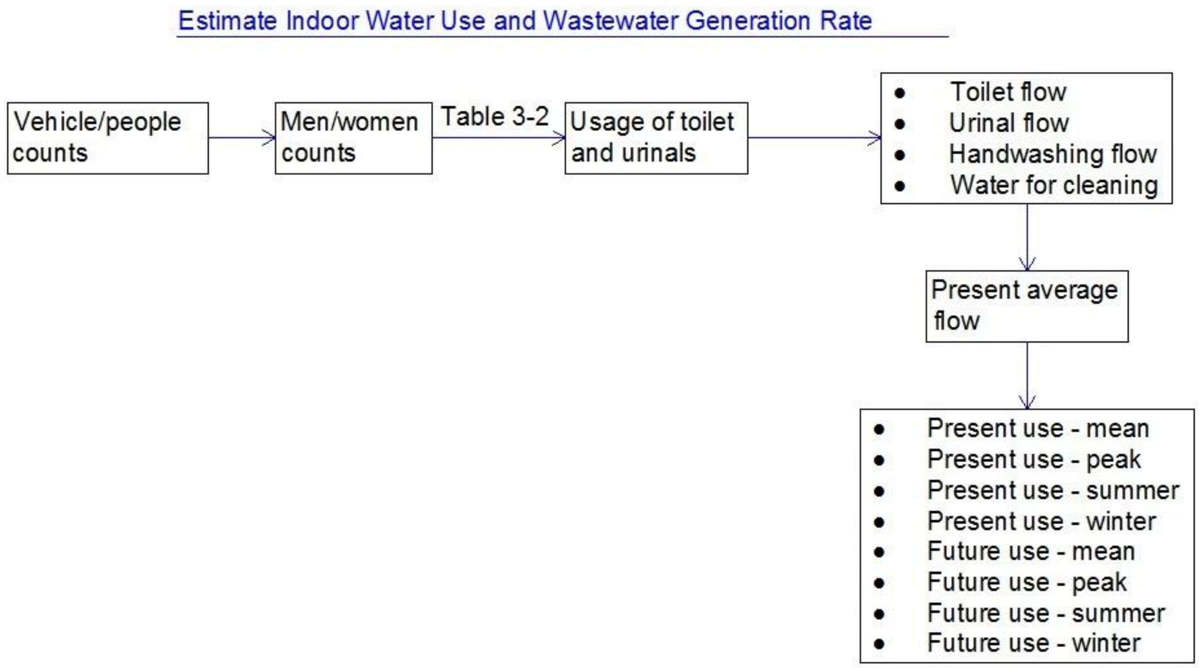


Chart 3-2
Estimation of indoor water use and wastewater generation rate flow chart

Example 1: Estimate of indoor water use and wastewater generation rate.

The ramp count at the Dunnigan-NB SRRA averages 1,100 veh/d. Using the data presented in Table 3-2, estimate the facility average and peak indoor water use and wastewater flowrate for 1.28 gal/flush toilets and waterless urinals. Assume the flush valve diaphragms are serviced on a regular basis.

Note that in this example, the relationship between traffic flow and people per vehicle has been established through site specific measurements. However, traffic data available for most SRRA facilities is not of sufficient quality to use for making design flow estimates. In general, the design engineer will need to review available SRRA usage data and determine the validity of the data set and, often, initiate the collection of supplemental usage data. Additional information of the collection of site specific usage data are presented in Sec. 3.4.

Solution

Note example was computed using a spreadsheet which may result in rounding variations.

1. Estimate the distribution of men and women using the SRRA data given in Table 3-2.
   a. Total people per day (pe/d) = (daily vehicle count) (people per vehicle) = (1100 veh/d) (1.3 pe/veh) = 1430 pe/d
   b. Men = (total people per day) (fraction of people that are men) = (1430 pe/d) (0.55) = 787 pe/d
   c. Women = (total people per day) (fraction of people that are women) = (1430 pe/d) (0.45) = 644 pe/d
2. Compute the fixture usage using the SRRA using the data given in Table 3-2.
   a. Men using urinals = (fraction of people that are men) (fraction of urinal use) = (787 pe/d) (0.8) = 629 pe/d
   b. Men using toilets = (fraction of people that are men) (fraction of toilet use) = (787 pe/d) (0.2) = 157 pe/d
   c. Women using toilets = (fraction of people that are women) (fraction of women using toilets) = (644 pe/d) (1) = 644 pe/d
3. Compute water use for fixture use and cleaning using the SRRA data given in Table 3-2.
   a. Toilet flow = (total number of men and women using toilets) [(nominal toilet flowrate) (flushes per person) (factor for flush valve fouling) + (urine contribution per person)] =
      (157 + 644 pe/d) [(1.28 gal/flush) (1.5 flush/pe) (1.0) + (0.055 gal/pe)] = 1582 gal/d
   b. Urinal flow = (number of men using urinals) (water use per urinal flush + urine contribution per user) =

(629 pe/d) (0 + 0.055 gal/pe) = 35 gal/d

c. Handwashing = (total people) (water use for hand washing per person) = (1430 pe/d) (0.125 gal/pe) = 179 gal/d

d. Water use for cleaning = 50 gal/d

e. Present average flow = (flow from toilets) + (flow from urinals) + (flow for handwashing) + (flow for cleaning) = 1845 gal/d

4. Estimate mean and peak water use for present (current) and future (30 years (y) from present) conditions using the SRRA given in Table 3-2.

   a. Present use, mean = present average water use from Step 3e = 1845 gal/d

   b. Present use, summer =
   (present average water use) (seasonal use factor) =
   (1845 gal/d) (1.16) = 2140 gal/d

   c. Present use, winter =
   (present average water use) / (seasonal use factor) =
   (1845 gal/d) / (1.16) = 1591 gal/d

   d. Present use, peak =
   (present average water use) (daily peaking factor) =
   (1845 gal/d) (2.0) = 3690 gal/d

   e. Future use, mean =
   (present average water use) (use escalation factor) =
   (1845 gal/d) (1.3) = 2399 gal/d

   f. Future use, summer =
   (average future water use) (seasonal use factor) =
   (2399 gal/d) (1.16) = 2783 gal/d

   g. Future use, winter =
   (average future water use) / (seasonal use factor) =
   (2399 gal/d) / (1.16) = 2068 gal/d

   h. Future use, peak =
   (average future water use) (daily peak factor) = (2399 gal/d) (2.0) = 4798 gal/d

Comment

As shown in this example, because the characteristic flowrate for SRRA facilities is primarily a function of the vehicle count, unit flowrates for individual fixtures, and usage patterns, great care should be used is collecting accurate usage data. In developing peak flow and future use estimates, consideration of the following local factors that could impact usage rates is recommended: highway expansion projects, parking limitations, local developments, and new infrastructure (e.g., high speed rail). In addition, flow input from other sources, such as recreational vehicle (RV) dump stations, heating, ventilation, and air conditioning (HVAC) systems, and water system backwash or blowdown flows will need to be considered in the water balance, if applicable.

3.2.3 Water Quality

The characteristics of the water supply can impact the design and operation of the wastewater treatment and water reuse systems. For example, water supply with low alkalinity can result in pH control problems, water supply with high sulfate can cause odor and concrete corrosion issues, and water supply with high dissolved solids can enhance the rate of metal corrosion. One consideration regarding the use of onsite water recycle systems is that there is less influence from the chemistry of the potable water supply as it is used as a makeup source to be blended with the product recycled water. It is still instructive to consider the chemical composition of the water supply. For example, the mass of alkalinity present contributes to the buffering of acid-producing chemical reactions, while pH, chlorine and hardness can impact the corrosion of pipes and longevity of flush valve diaphragms.

Analytical data from several example water supplies are summarized in Table 3-3. The parameters in Table 3-3 also represent the recommended minimum water quality characterization for the water supply.

Table 3-3
Water supply data from Gaviota, Erreca, Shandon, and Dunnigan SRRAs

		SRRA facility			
Parameter	Unit	Dunnigan	Erreca	Gaviota	Shandon
Source	–	Groundwater	Aqueduct	Spring	Groundwater
TDS	mg/L	230	220	570	1612
pH	–	8.2	8.1	7	6.6
Cl^-	mg/L	5	73	39	–
SO_4^{2-}	mg/L	2	18	139	–
Alkalinity	mg/L as $CaCO_3$	210	63	280	220
NO_3^-	mg/L	9	–	–	<0.01
Hardness	mg/L	170	79	402	580

3.3 WASTEWATER DATA

The characteristics of SRRA wastewater are different from those of most other types of wastewater because there is little dilution water blended with human waste. A large amount of wastewater flow and quality data were collected at the Dunnigan SRRA and other SRRA facilities throughout California, including data from traffic counters, people counters, and logging water meter installations. The information presented in this section is based on the results of these studies.

3.3.1 Wastewater Flowrates

The characteristic flowrate for SRRA facilities is primarily a function of the unit flowrates for individual fixtures and usage patterns. An estimate of the wastewater flow can be obtained from water meters used to measure how much water is used indoors. In some cases, water meters are not installed, or are installed in a location that measures water used indoors and outdoors. It is therefore important for the design engineer to inspect the flowmeter to confirm the water stream that is being metered prior to using flow estimates based on the water meter as the basis for process design.

As summarized on Fig. 3-4, the average flowrate (50 percent) observed at most SRRAs ranges from 2,000 to 7,000 gal/d, and the peak flowrates (99 percent) range from 10,000 to 20,000 gal/d. Note that all the flow data presented as a log-normal probability distribution on Fig. 3-4 have essentially the same slope, which is an indication that water use patterns are subject to similar patterns of variability. Further, design wastewater flowrates can be predicted readily at other facilities using basic parameters and assuming the nature of the distribution is as shown on Fig. 3-4.

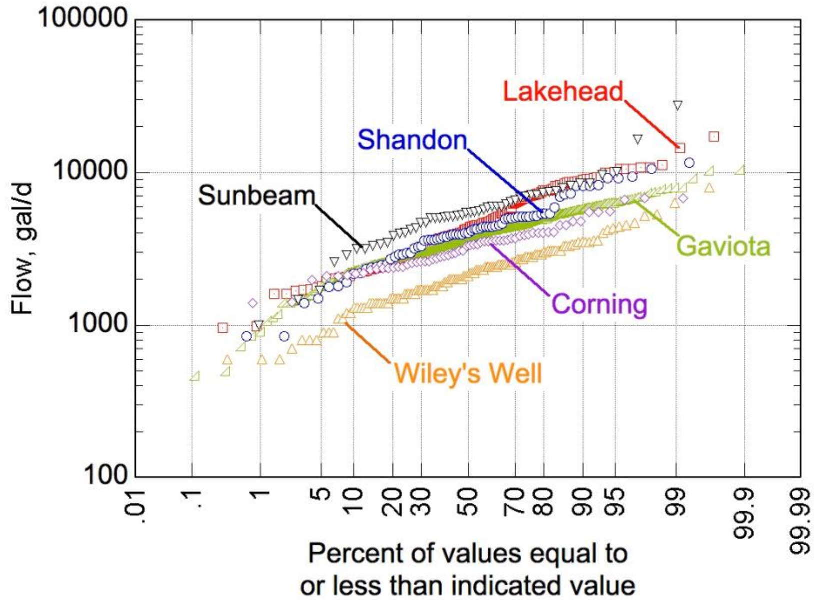


Figure 3-4
Historical water use at selected SRRA facilities
(Source: Caltrans Division of Engineering Services,
Water and Wastewater Engineering Branch)

3.3.2 Wastewater Characteristics

The characteristics of wastewater at SRRA facilities are primarily a function of the chemical characteristics of the water used for flushing and cleaning activities and the substances added to the water during use. Unlike domestic and municipal wastewater, SRRA wastewater is highly concentrated because it is derived from toilet and urinal flushing and is not diluted by greywater sources typical of household wastewater.

A comparison of raw wastewater concentrations for typical domestic wastewater and Dunnigan SRRA wastewater is presented in Table 3-4. As reported in Table 3-4, the wastewater obtained from SRRA facilities can be considered high strength in terms of oxygen demand and nitrogen, when compared with domestic wastewater. The SRRA wastewater is more concentrated because it does not include significant flows from greywater sources, which are typical of domestic water use. Further, SRRA facilities receive a large volume of urine, which is a concentrated source of nitrogen.

Table 3-4
Comparison of raw wastewater from Dunnigan SRRA with typical domestic wastewater

Constituent	Unit	Domestic wastewater[a]	Dunnigan SRRA
Biochemical oxygen demand	mg/L	210	470
Chemical oxygen demand	mg/L	500	1600
Total alkalinity	mg/L	120	1150
Ammonium as N	mg/L	22	245
Total hardness	mg/L	20	330
pH	pH units	7.5	7.4
Total Kjeldahl nitrogen[b]	mg/L	35	280
Phosphorus, total, as P	mg/L	7	29
Total volatile solids	mg/L	360	745
Total suspended solids	mg/L	210	550
Total dissolved solids	mg/L	500	1000
Total coliform	MPN/100mL	10^7	10^8
Fecal coliform	MPN/100mL	10^6	10^7

a. Adapted from Crites and Tchobanoglous (1998)

b. Total nitrogen not included because Total Kjeldhal nitrogen is equal to nitrogen in raw wastewater.

3.3.3 Constituent Mass Loading

The raw wastewater characteristics can be modeled using per-person constituent loading data and the water usage estimates. Constituent mass loading data were developed in the Dunnigan SRRA pilot study from grab samples of the waste stream collected from the discharge of the raw waste grinder lift station. Using the measured constituent concentrations, facility usage data from people count devices and water use data were analyzed to develop the data presented in Table 3-5. Note that the data summarized in Table 3-5 should be considered as preliminary in projecting waste stream constituent loading estimates. Other factors that can impact the waste stream characteristics include chemistry of the water supply, constituents recycled where water reuse is practiced, and chemicals added during water treatment or comfort station maintenance.

Table 3-5
Mass loading parameters observed at NB Dunnigan SRRA

	Mass loading, g/pe[a]	
Constituent	Range	Typical
cBOD	1.765 - 4.261	2.6
COD	3.833 - 11.747	6.8
sCOD	1.551 - 3.226	2.4
DOC	0.547 - 1.045	0.83
TOC	0.687 - 2.023	1.3
Surfactant	0.001 - 0.008	0.004
TS	5.688 - 12.894	7.9
TDS	3.822 - 9.937	5.3
TSS	1.102 - 4.553	2.5
VSS	0.967 - 4.238	2.2
Hardness as $CaCO_3$	2.068 - 2.293	2.2
Alkalinity as $CaCO_3$	4.429 - 5.823	5.1
NH_4-N	0.749 - 0.906	0.83
NO_2-N	0.001 - 0.013	0.00
NO_3-N	0.001 - 0.029	0.01
org N	0.29 - 0.681	0.44
TKN	1.105 - 1.529	1.28
TN	1.134 - 1.536	1.30
Ortho-P	0.062 - 0.099	0.074
org P	0.016 - 0.076	0.036
TP	0.078 - 0.175	0.11

a. g/pe = gram per person per usage

cBOD = carbonaceous biochemical oxygen demand; sCOD = soluble chemical oxygen demand; DOC = dissolved organic carbon; TOC = total organic carbon; TS = total solids; VSS = volatile suspended solids

3.4 DEVELOPMENT OF DESIGN DATA

The basic design data required for starting a recycled water system design can be developed using historical records, collecting current usage data, comparing with other facilities, and making assumptions based on past experience. Based on the discussion presented above, a projected wastewater composition can be computed for a typical SRRA facility.

The people count data are used to assess the total constituent loading, and when they are considered along with the water use data, expected constituent concentrations can be computed. Key design criteria and related considerations are presented in Table 3-6. The engineer should consider the factors in Table 3-6 in developing design usage rates.

Table 3-6
Summary of preliminary design data collection and related considerations for water reuse systems

Design criteria	Considerations	Data Collection
Vehicle counts	Daily peaks occur between the hours of 9am and 5pm Weekly peaks usually take place on Fridays, Sundays, and during holiday travel periods Seasonal peaks take place in the summer Tour buses can result in peak hourly usage Parking limitations limit the maximum number of SRRA visitors at some locations Various traffic logging systems are available that automatically record usage data Traffic loggers are typically placed in a subsurface enclosure near the exit ramp	 Traf-X counter device placed near exit ramp for SRRA. Computer is connected to device to download historical data.
People counts	Fraction of men and women can be established by placing people counters near doorways A cage over the logger can deter theft of the sensor Peak usage is a function of summer travel, holiday travel, and tourist seasons Data collection for a full year is recommended, but several weeks of monitoring in the summer and winter can be used to approximate the annual usage using the seasonal variation factors from Table 3-2 and the procedure shown in Example 1.	 SenSource infrared sensor placed above SRRA doorway. Data are sent wirelessly to logging unit.
Water usage	Fixture flowrates have a significant impact on per person water use Leaking and fouled flush valves tend to increase water use and sometimes experience catastrophic failure Faulty sensors result in multiple flush events Using multiple flow meters, data logging water meters can be used to track water usage for separate water uses, such as handwashing, toilet flushing, and irrigation Manual readings can be used if available 7 days per week (d/wk) Need to confirm that meter readings do not include irrigation water use Meter output signals include pulse, voltage, and current (4-20 mA); need to check compatibility of signal with logging or SCADA system	 Seametrics flow meter with integrated reed switch and output to data logging system

Table 3-6 (Continued)
Summary of preliminary design data collection and related considerations for water reuse systems

Design criteria	Considerations	Data collection
Concentration	Constituent concentrations are relatively consistent for a given facility Higher concentration from men due to reduced water use for urinal flushing Challenging to collect composite samples for raw wastewater New wastewater systems should be constructed with sampling locations to facilitate process monitoring.	 Water quality can be determined by grab sampling of water and wastewater systems.
Water temperature	Temperature impacts process reaction rates for most biological processes Sensors can be suspended in water and wastewater systems to obtain detailed water temperature data Temperature in wastewater system varies seasonally and depends on source and temperature of water supply	 Temperature logger by Onset Computer Co.

Trend data that were collected at the Dunnigan-NB SRRA are shown on Fig. 3-5. By observation, it is apparent that there is a seasonal variation in usage patterns, with the summer season defined as April through August, and the winter season defined as September through March. As shown in Ex. 1, there is a direct correlation between the flow of traffic, people, and water use. Therefore, the people count data can be used to accurately estimate flowrates.

Typical hourly variations in flow are presented on Fig. 3-6 for the O'Brien SRRA. As shown on Fig. 3-6, nearly all of the flow occurs over a 10- to 12-h period. Therefore, the peak hourly flow can be determined by dividing the average daily flow by 10 h.

Water temperature (generally recorded from the primary tank) has an impact on the performance of anaerobic and aerobic treatment processes. As will be shown subsequently, the rate of biological treatment processes is a function of temperature. In general, the design process should consider both seasonal loading and seasonal impacts of temperature on biological processes to determine what factor controls the design. The peak wastewater temperature is expected to occur from July through September in the range of 85 degrees Fahrenheit (F). The minimum wastewater temperature is expected to occur from December through February in the range of 60 degrees F.

Because of the size of California, a wide range of possible climates exist where water reuse may be used. Selected temperature profiles for SRRA facilities in California are presented on Fig. 3-7 and can be used for estimating the range of expected temperatures. Temperature data are shown for northern (see Figs. 3-7a,b), central (see Figs. 3-7c,d,e), and southern regions (see Fig. 3-7f).

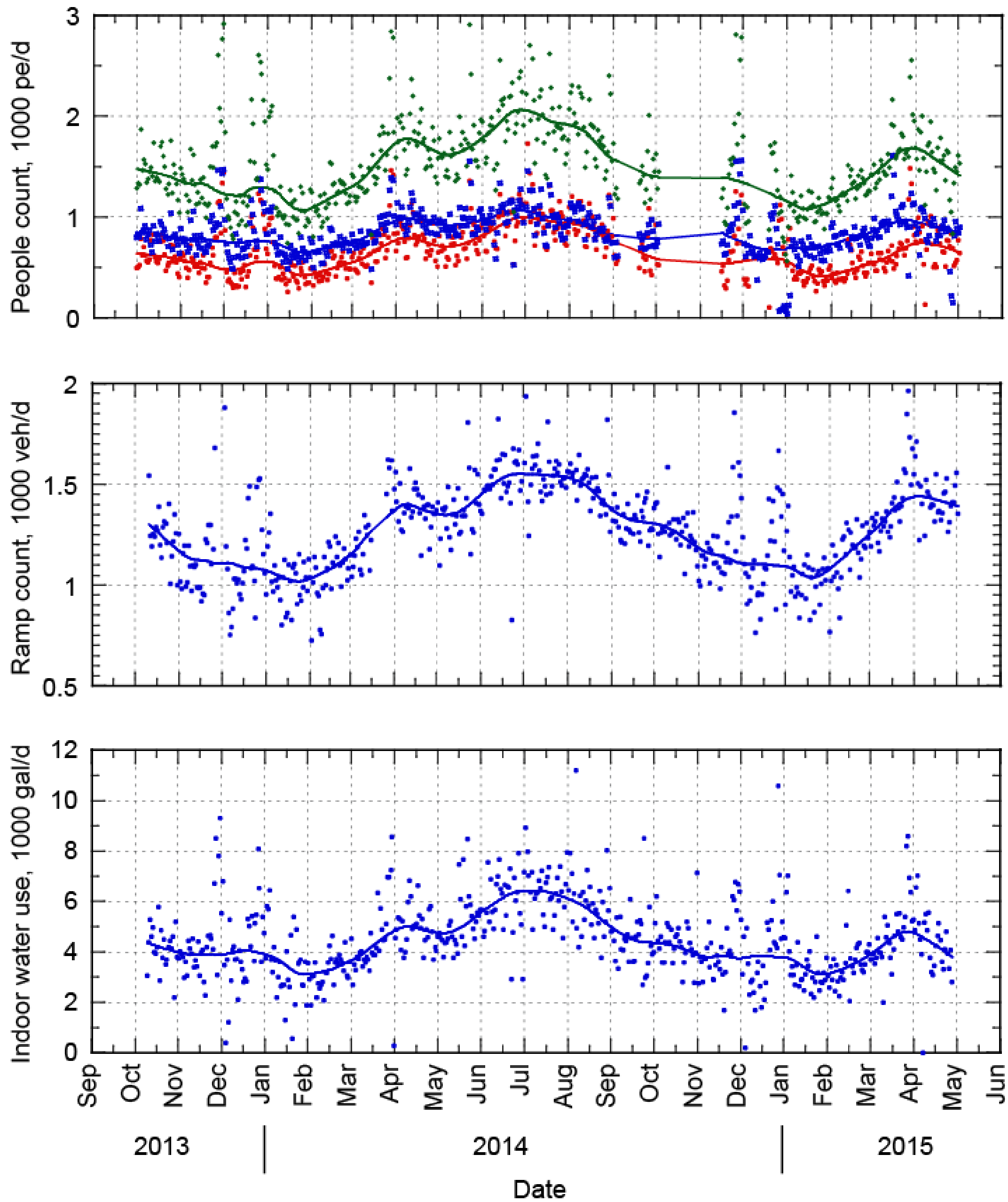


Figure 3-5
Summary of usage data collected at Dunnigan-NB SRRA

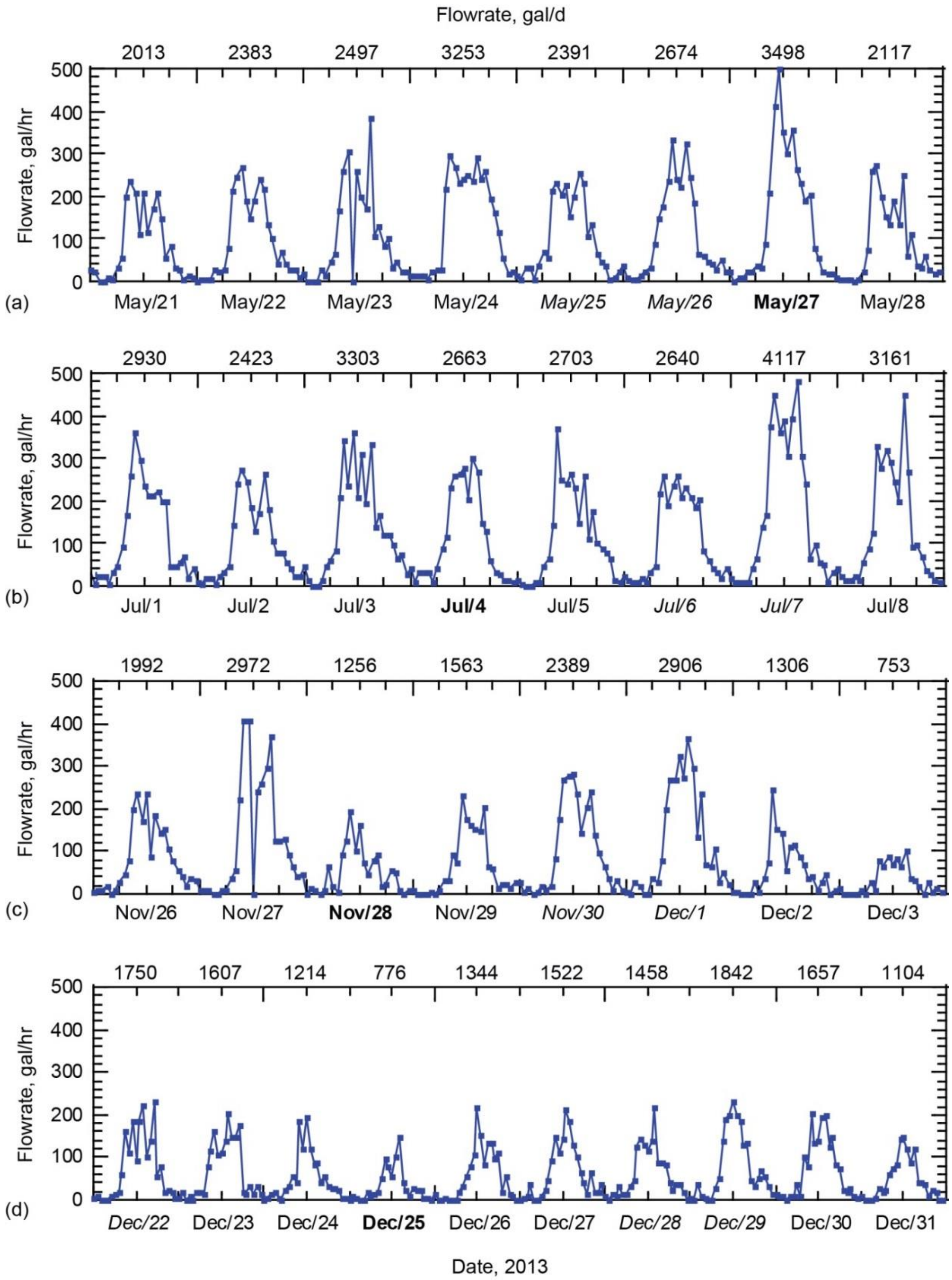


Figure 3-6
Flow trends at O'Brien SRRA.
Note that the dates shown on horizontal axis are coded as follows: weekend days are italicized, holidays are bold, and non-holiday weekdays are normal font.

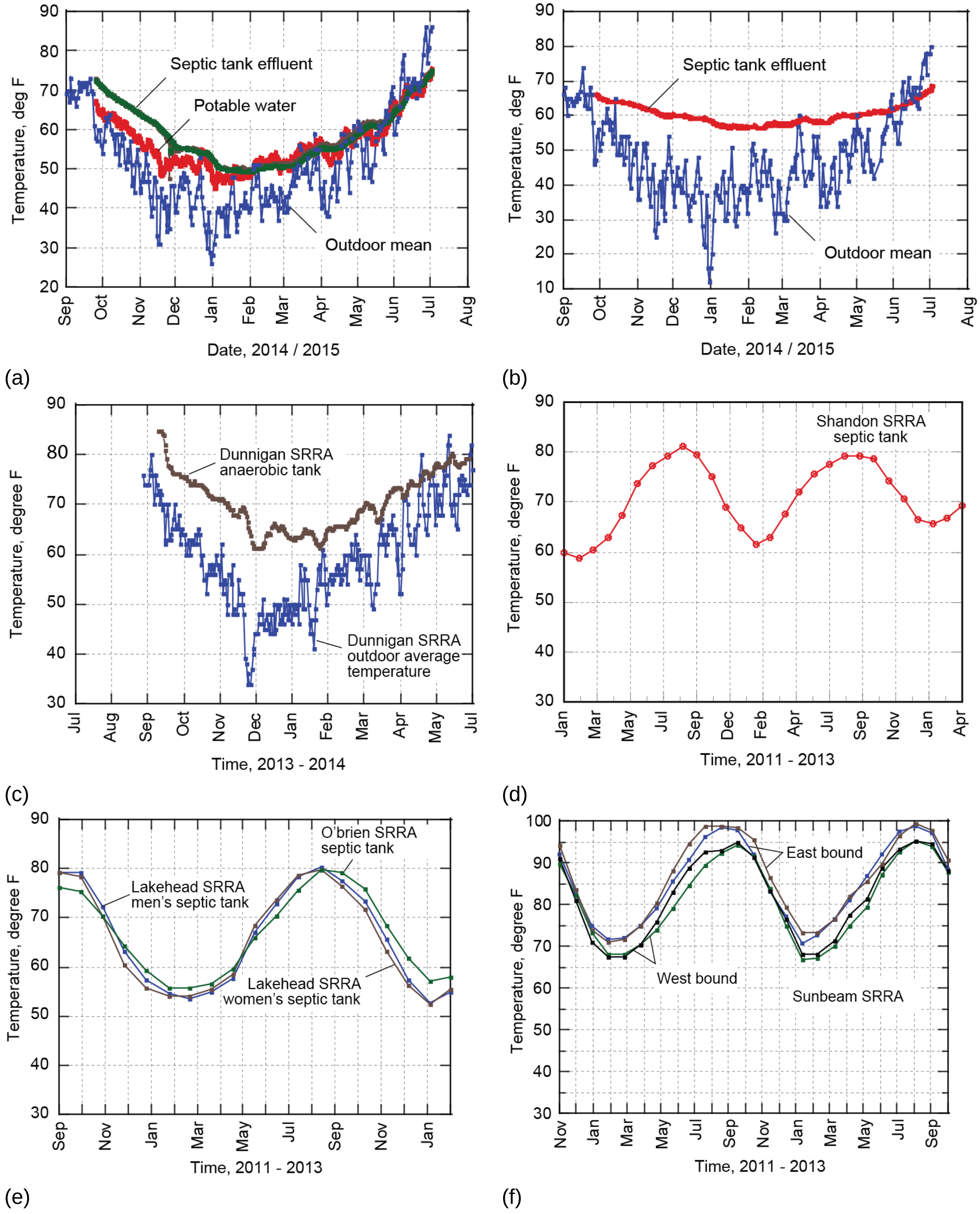


Figure 3-7
Temperature profiles at SRRA facilities throughout California:
(a) RE Collier, (b) Honey Lake, (c) Dunnigan, (d) Shandon, (e) Lakehead, (f) Sunbeam

The procedure to compute design loading parameters is summarized on Chart 3-3, and demonstrated in Ex. 2.

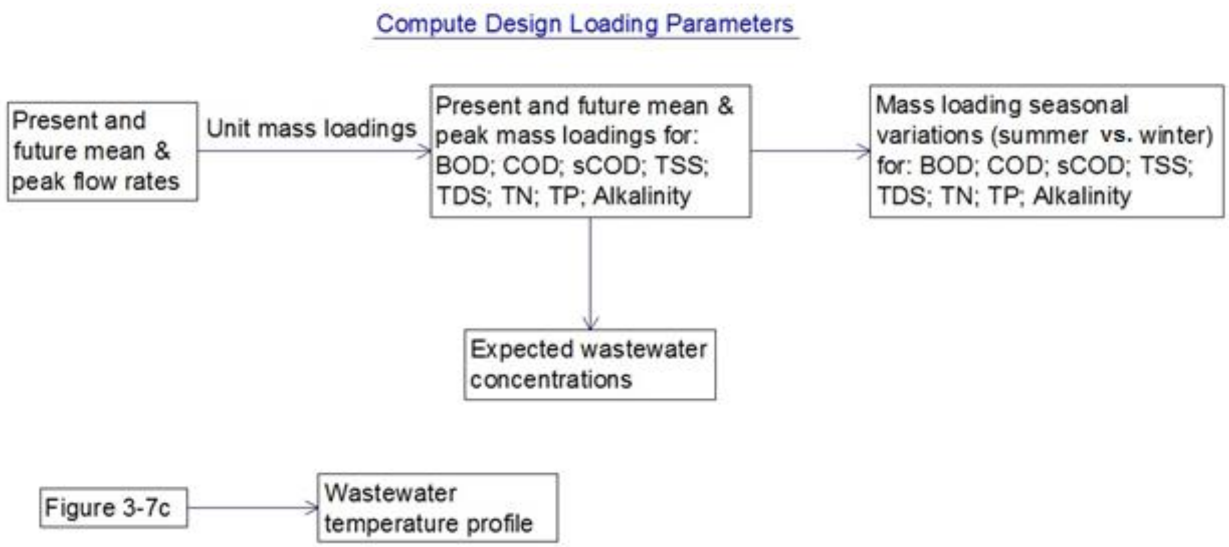


Chart 3-3
Computation of design loading parameters flow chart

Example 2: Compute design loading parameters.

Water reuse is being considered for the SRRA facility located in the Central Valley region and described in Ex. 1. Using the data findings in Ex. 1 and the unit mass loading data given in Table 3-5, estimate the facility mass loading and expected constituent concentrations for BOD, chemical oxygen demand (COD), soluble chemical oxygen demand (sCOD), TSS, TDS, TKN, $NH4^+$-N, total phosphorus (TP), and alkalinity. Effects of the expected wastewater temperature are also described.

Solution

1. Estimate mean and peak people counts for present and future conditions using the SRRA data given in Table 3-2.
   a. Present use, mean = from Ex. 1, Step 1 = 1430 pe/d
   b. Future use, mean = (present mean people per day) (usage escalation factor) = (1430 pe/d) (1.3) = 1859 pe/d

c. Present use, peak = (present mean people per day) (daily peaking factor) = (1430 pe/d) (2.0) = 2860 pe/d

d. Future use, peak = (future mean people per day) (daily peaking factor) = (1859 pe/d) (2.0) = 3718 pe/d

2. Set up a computation table for the mass loading data.

Constituent	Unit mass loading, grams per person (g/pe)	Mass loading, grams per day (g/d)			
		Present		Future	
		Mean	Peak	Mean	Peak
BOD	2.56	3661[a]	7322	4759	9518
COD	6.76	9667	19334	12567	25134
sCOD	2.43	3475	6950	4517	9035
TSS	2.46	3518	7036	4573	9146
TDS	5.31	7593	15187	8971	19743
TN	1.30	1859	3718	2417	4833
TP	0.11	157	315	204	409
Alkalinity	5.14	7350	14700	9555	19111

a. BOD = (present mean people per day) (unit mass loading) = (1430 pe/d) (2.6 g/pe) = 3718 g/d

3. Set up a computation table for the seasonal variation in mass loading.

Constituent	Mass loading, g/d			
	Present		Future	
	Summer	Winter	Summer	Winter
BOD	4247[a]	3156	5520	4103
COD	11213	8333	14578	10833
sCOD	4031	2996	5240	3894
TSS	4081	3033	5305	3942
TDS	8808	6546	11451	8510
TN	2156	1603	2803	2083
TP	182	136	237	176
Alkalinity	8526	6336	11084	8237

a. BOD = (present mass loading from Step 2) (seasonal use factor from Table 3-2) = (3661g/d) (1.16) = 4247 g/d

4. Set up a computation table for the expected wastewater concentration data.

Constituent	Concentration, mg/L
BOD	524[a]
COD	1384
sCOD	498
TSS	504
TDS	1087
TN	266
TP	23
Alkalinity	1053

a. BOD = [(present mean mass loading) (conversion gram to milligram)] / [(present mean water use) (conversion of gallon to liter)] = [(3661 g/d) (1000 mg/g)] / [(1845 gal/d) (3.785 L/gal)] = 524 mg/L

mg/g = milligrams per gram; L/gal = liters per gallon

5. Estimate the temperature profile using Fig. 3-7c for the Dunnigan SRRA, which is located in the Central Valley region.
   a. The peak wastewater temperature is expected to occur July through September in the range of 85 degrees Fahrenheit (F).
   b. The minimum wastewater temperature is expected to occur December through February in the range of 60 degrees F.

Comment

In this case, the minimum and maximum temperatures correspond with the period of minimum and maximum use, respectively. Also, note that the concentration computation applies to all scenarios because the loading and flowrate are assumed to be proportional. The mass loading data can be differentiated for summer and winter season loading.

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4.0 SYSTEM DESIGN OVERVIEW

Prior to developing a detailed design for a project, an advanced planning study (APS) is used by Caltrans to evaluate the adequacy of existing facilities and prepare a preliminary project design. The APS should include a preliminary design to size and select the process tanks and equipment. An approach for developing a preliminary system design is outlined in this section, as shown in Chart 4-1.

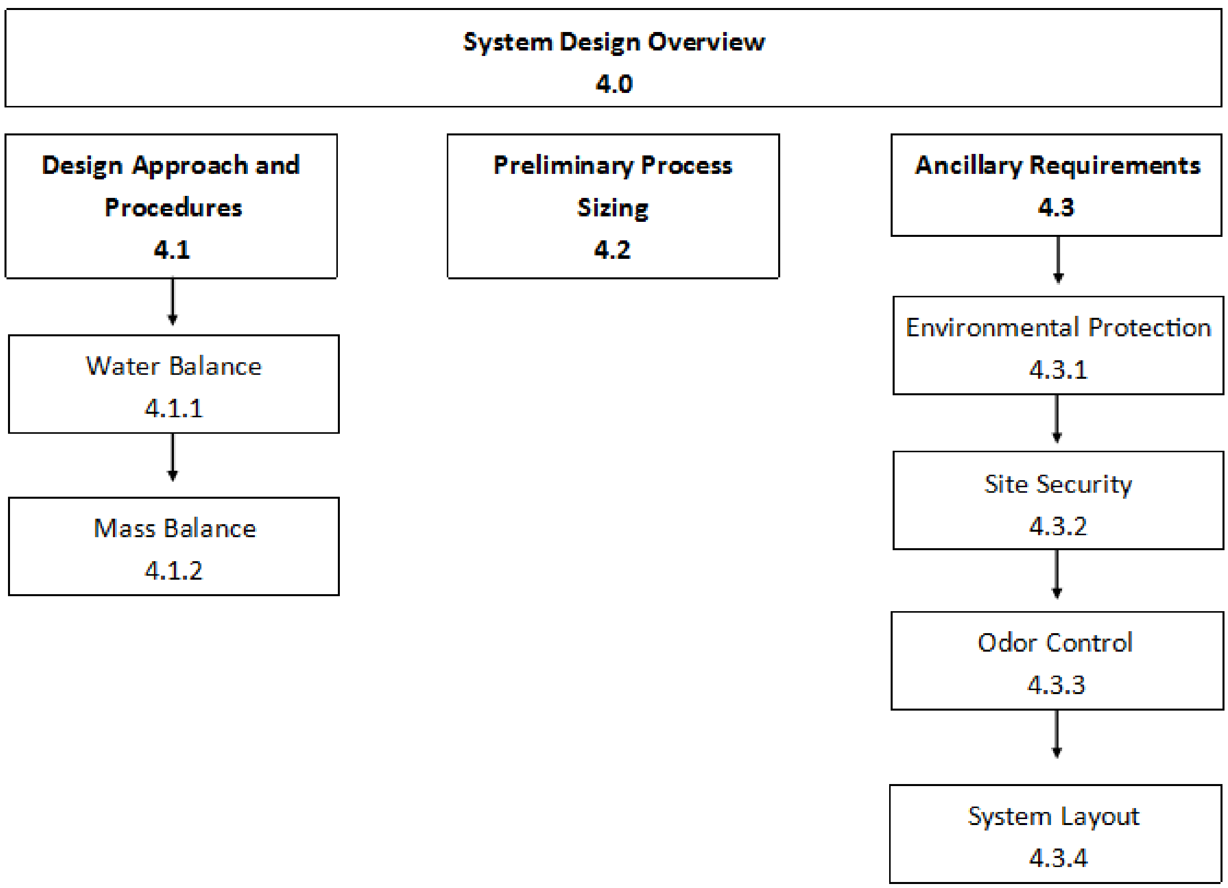

Chart 4-1
Preliminary system design steps

An overview of the process flow diagram developed on the basis of the study conducted at the Dunnigan SRRA pilot facility is shown on Fig. 4-1.

The processes presented in this section were validated at the Dunnigan SRRA pilot facility, but other processes can be used as alternatives. For example, biological treatment can be achieved using any one of hundreds of technologies, including membrane bioreactor, textile-based filter, or recirculating sand filter. Each technology has specific requirements for footprint, operation, and maintenance. The processes presented in this section were selected for balance between these considerations. The assumptions behind the process treatment train presented in this section are (1) limited availability of space, (2) minimal operation and maintenance needs, (3) use of off-the-shelf components to the extent possible, and (4) compliance with applicable water recycling criteria.

4.1 DESIGN APPROACH AND PROCEDURES

The basic approach to developing an initial design is to use preliminary design values and relationships that closely approximate the final design. The design approach begins with a review of data collected from onsite data logs and/or models. The data can be validated by obtaining agreement between data logs, models, and results from other facilities. After the data have been validated, a system schematic diagram and mass balance analysis are used, along with typical design values, to define the system components. Spreadsheet solutions are helpful for mass balance computations. Number of tanks and filters is based on the Dunnigan pilot study, but may be adjusted up or down based on site-specific conditions.

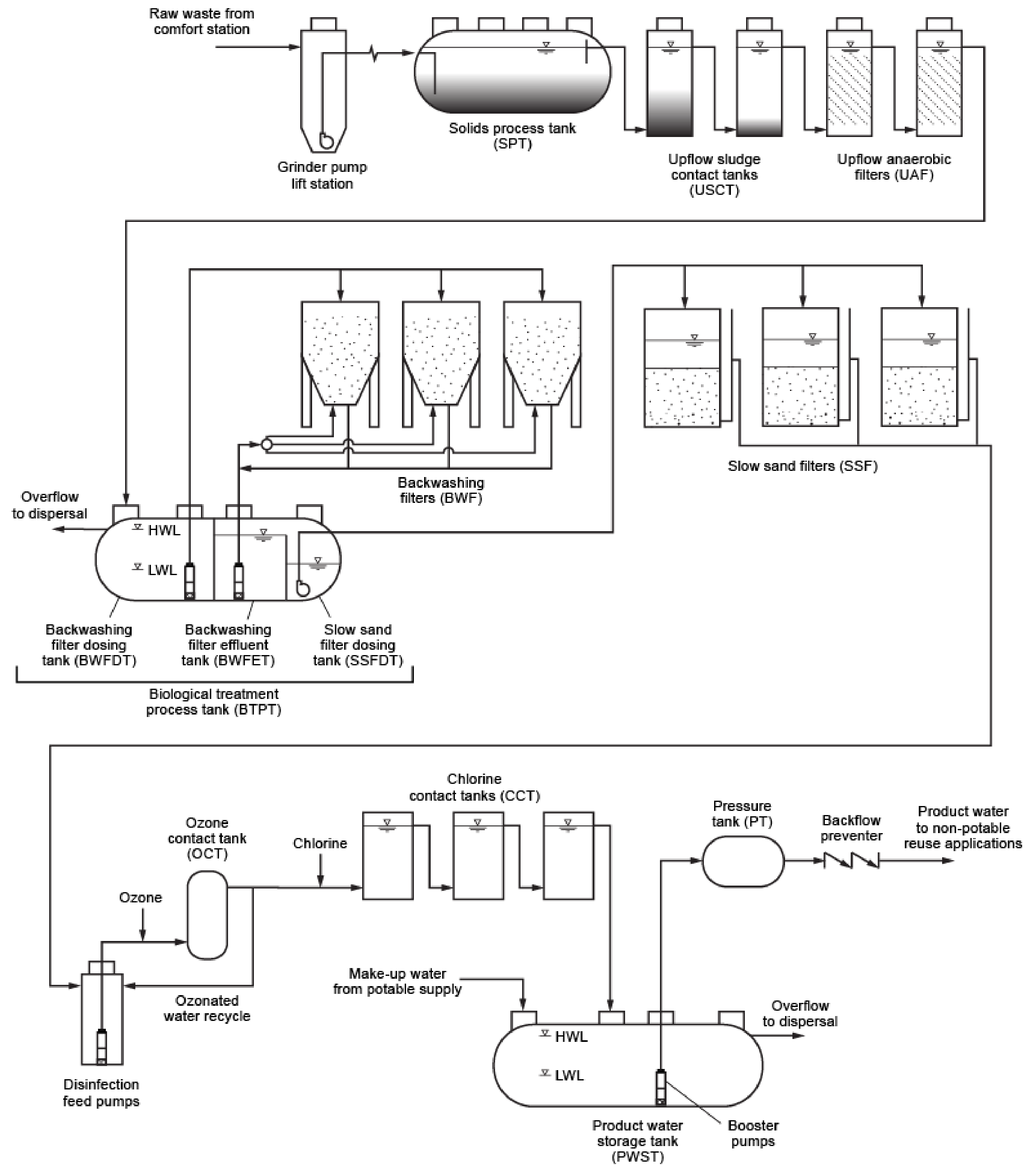

Figure 4-1
Overview of water recycle treatment train showing typical process configuration.
The diagram corresponds approximately to the design procedure presented in Ex. 3.

4.1.1 Water Balance

The purpose of the water balance is to determine the amount of water that can be recycled for a given facility. As described in Sec. 3, water is used for potable and nonpotable applications at the SRRA. Potable applications include drinking fountains, hand washing, cleaning, and jug filling; nonpotable applications include irrigation, toilet flushing, and urinal flushing. For purposes of the analysis presented in this manual, the water balance is limited to water used indoors that enters the drainage system, which includes water used for hand washing, cleaning, and flushing. The design water reuse rate can be estimated using Eq. (4-1).

$$W_{Ri} = (W_{NPi} / W_{Ti})\,(100) \tag{4-1}$$

where W_{Ri} = water recycle rate, percent
W_{NPi} = indoor nonpotable water use, gal/d
W_{Ti} = indoor total water use, gal/d

Note that the computation based on Eq. 4-1 corresponds to the maximum rate of water reuse that is possible. To achieve maximum water reuse, it will be necessary to have flow equalization, which is discussed in Sec. 7.

4.1.2 Mass Balance

To determine the chemical characteristics of the recycled water and the water discharged to onsite dispersal, it is necessary to account for mass transformations. The mass loading dataset developed in Sec. 3 is the starting point for the mass balance analysis, and represents the average loading condition. For purposes of this analysis, the constituent transformation is divided into three categories (1) biodegradable organics and suspended solids, (2) nitrogen and alkalinity, and (3) conservative materials.

Biodegradable organics are measured as carbonaceous BOD (cBOD), the total organic load including nonbiodegradable fraction is reported as COD, and suspended solids are measured as TSS. Proper functioning of the water recycling system requires nearly complete (i.e., greater than 95 percent) transformation of these constituents. Nitrogen is considered separately because the transformations of organic and ammonium nitrogen to nitrate and nitrogen gas are more complex than simple removal. Furthermore, the transformation of nitrogen is significantly affected by the presence of alkalinity, and in the absence of sufficient alkalinity to buffer the process pH, the conversion of ammonium to nitrate is inhibited. Substances such as salts, minerals, and nonbiodegradable organic compounds are known as “conservative materials” because they are not affected significantly by the treatment system. In general, conservative materials can be removed using membrane technologies such as reverse osmosis, electrodialysis, and advanced oxidation. For practical purposes, TDS and phosphorus are effectively conservative materials because the treatment process does not remove very much of them. Expected removal rates, based on performance observed at the Dunnigan SRRA pilot facility, are summarized in Table 4-1.

Table 4-1
Summary of expected design constituent removal for process treatment train

	Percentage removed		
Constituent	Anaerobic	Aerobic	Overall
BOD	60	98	99
COD	50	97	98
TSS	80	99	99
TDS	NR	NR	NR
Nitrate-N	99	–	–
Ammonium-N	–	99	–
TP	NR	NR	NR

NR = not removed

4.2 PRELIMINARY PROCESS SIZING

The initial site layout and process configuration for purposes of planning studies can be based on preliminary process sizing criteria for the average loading and site conditions expected. Several assumptions must be made for application of the sizing criteria, including (1) flow equalization, (2) maximum recycle rate, and (3) a design based primarily on average loading and site conditions. Subsequent refinements to the design will take into consideration factors such as seasonal peak loading, diurnal variations, temperature fluctuations, and modular designs. Based on site specific limitations the system sizing may be increased or decreased. Selected criteria that can be used for the preliminary design of the key process components are shown on Fig. 4-1 and are described in Table 4-2. The application of these design criteria is demonstrated in Ex. 3.

Table 4-2
Summary of preliminary sizing for key process components

Element	Description	Preliminary sizing criteria
SPT	A solids process tank (SPT) is used to remove trash, debris, suspended organic solids, and some soluble organic matter. It also serves as primary denitrification, solids storage, and solids digestion tank. An SPT should be easy to access to facilitate tank maintenance activities.	1-d hydraulic retention time (HRT) at projected future mean flow Assume 1/3 of tank volume is for solids storage
USCT	An upflow sludge contact tank (USCT) is used to remove fine solids that carry over from the SPT and pass the wastewater flow through a bed of active anaerobic biomass that accumulates at the bottom of the tank.	Upflow velocity should not exceed 1 meter per hour (m/h) at maximum hourly peak flow Liquid depth = 2 meters (m)
UAF	An upflow anaerobic filter (UAF) is used to remove additional dissolved organic matter by providing surface area for anaerobic bacteria to colonize and contact wastewater as it progresses through the filter tanks. In general, two filters in series are recommended on the basis of research to date. Alternative designs may use additional filters.	Upflow velocity should not exceed 1 m/h at maximum hourly peak flow Liquid depth = 2 m
BTPT	A biological treatment process tank (BTPT) consists of three compartments: backwash filter dosing tank (BWFDT), backwash filter effluent tank (BWFET), and slow sand filter dosing tank (SSFDT). The BWFDT must be large enough to provide full flow equalization to maximize water reuse. The BWFET is used to store effluent that is later used for filter backwash operations. The SSFDT has extra capacity to manage head decant during filter maintenance.	BTPT working volume should be equal to the expected future summer flow BWFDT is sized at 1500 gal SSFDT is sized at 2000 gal
BWF	A backwashing filters (BWF) is used for primary COD removal and conversion of ammonium to nitrate. The BWFs are designed such that all filters are dosed simultaneously, but backwash operations take place sequentially and typically backwash once per day. Backwash is conducted with BWF effluent and backwash waste is discharged to the SPT.	Nitrification capacity = 650 g N/d per filter (A =4 ft x 4 ft) Dosing frequency is 720 dose/d Backwash frequency is 1/d
SSF	Slow sand filters (SSFs) are designed to remove fine particles and must periodically be cleaned manually (i.e., every 60 d). The head is drained back to the SSFDT at the time of cleaning.	Hydraulic loading rate = 40 gallons per square foot per day (gal/ft^2•d), based on future mean flow Cleaning frequency is ~60 d
OCT	An ozone contact tank (OCT) is used to react ozone with compounds that color the water. A portion of the ozonated water is recycled through for additional ozone injection as needed to remove color.	Assumed ozone demand is 50 mg/L with 1-min HRT Design future summer flow Typical flow is 10 gal/min

Table 4-2 (Continued)
Summary of preliminary sizing for key process components

Element	Description	Preliminary sizing criteria
CCT	A chlorine contact tank (CCT) provides holding time for chlorine to react with constituents in the water. In general, the pretreatment is used to remove interfering constituents so that the chlorine use is focused on microbial constituents.	Minimum is 3-h HRT Flowrate is mean flow divided by 24 h Chlorine dose ~ 15 mg/L
PWST	A product water storage tank (PWST) should be sized similar to the BWFDT, such that the product water flow is fully equalized to compensate for variable use.	PWST working volume is equal to the expected future summer flow
PT	A pressure tank (PT) is used to maintain hydraulic pressure for actuation of flush valves. A static pressure in the range of 60 to 70 pounds per square inch (lb/in^2) is desirable, and a flushing pressure of 30 to 40 lb/in^2 will allow proper operation of flush valves and toilets.	Nominal PT size is 1000 gal Static pressure is 60 to 70 lb/in^2

The design procedure for preliminary process sizing is summarized on Chart 4-2, and demonstrated in Ex. 3.

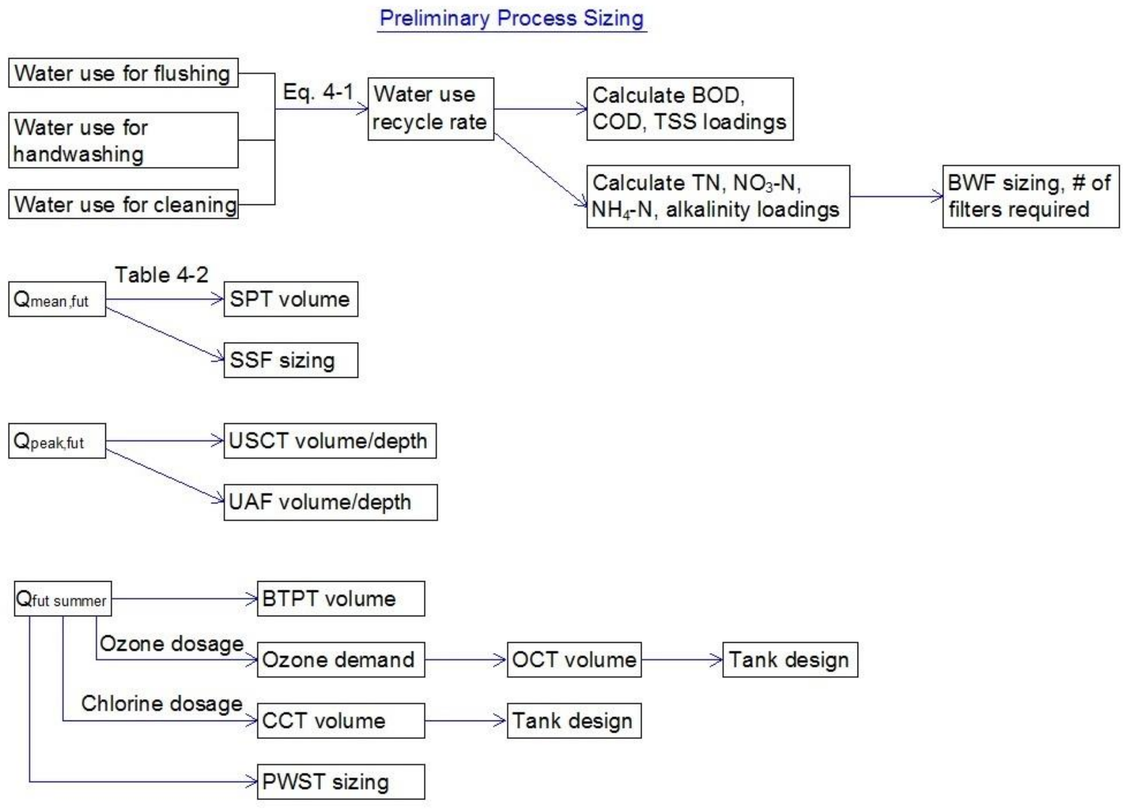

Chart 4-2
Computation of preliminary process sizing flow chart

Example 3: Compute preliminary process sizing.

The SRRA facility in the Central Valley region (as described in Ex. 1; see Sec. 3) is being considered for water reuse. Using the information presented in Ex. 1 and the mass loading data derived in Ex. 2 (see Sec. 3), estimate the facility recycle rate, the constituent concentrations of the system effluent (recycled water), and preliminary process sizing. Ignore seasonal temperature fluctuations.

Solution

1. Determine the water recycle rate using Eq. (4-1), Table 3-2, and the results from Ex. 1.
   a. Recycled water use for toilet flushing = (total number of men and women using toilets) [(nominal toilet flowrate) (flushes per person) (factor for flush valve fouling)] = (157 + 644 pe/d) [(1.28 gal/flush) (1.5 flush/pe) (1.0)] = 1538 gal/d (Note: If water flush urinals are used, the flowrate for urinals should be included.)
   b. Handwashing = (total people) (water use for hand washing per person) = (1430 pe/d) (0.125 gal/pe) = 179 gal/d
   c. Water use for cleaning = 50 gal/d (assumed)
   d. WRi = (W_{NPi} / W_{Ti}) (100) =
      (water use for flushing) / (water for flushing, hand washing, and cleaning) = ([(1538 gal/d) / (1538 gal/d + 179 gal/d + 50 gal/d)] 100 = 87%.
2. Compute effluent constituent concentrations for organics and suspended solids using a mass balance approach. The flow will be equalized fully, so use mean mass loading.

Constituent	Influent mass loading, g/d	Process effluent mass loading, g/d		Process effluent concentration, mg/L
		Anaerobic	Aerobic	
BOD	3718	1487[a]	30	4[b]
COD	9724	4862	146	21
TSS	3575	715	7	1

a. BOD, anaerobic effluent = (BOD influent mass loading) (1 – BOD removal in anaerobic process) = (3718 g/d) (1 – 0.6) = 1487 g/d.

b. BOD, effluent = (BOD process effluent mass loading) (conversion of g to mg) / [(average flowrate from Ex. 1) (conversion of gal to L)] = (30 g/d) (1000 mg/g) / [(1845 gal/d) (3.785 L/gal)] = 4 mg/L.

3. Compute effluent constituent concentrations for nitrogen and alkalinity using a mass balance approach.

Constituent	Influent mass loading, g/d	Process effluent mass loading, g/d		Effluent concentration, mg/L
		Anaerobic	Aerobic	
TN	1859	1859	1859	277
NO_3-N	—	0[a]	1859	277
NH_4-N	—	1859	0[b]	0
Alkalinity	7293	14,450[c]	1251[d]	186

a. Assume full denitrification.

b. Assume full nitrification.

c. Alkalinity is recovered at 3.85 g/g-nitrate-N removed (see Sec. 6).
Alkalinity, anaerobic effluent = (alkalinity influent mass loading) + (nitrate, aerobic process effluent mass loading) (3.85 g/g nitrate removed) = (7293 g/d) + (1859 g/d) (3.85 g/g) = 14,450 g/d

d. Alkalinity is consumed at 7.1 g/g-ammonium-N oxidized (see Sec. 7).
Alkalinity, aerobic effluent = (Alkalinity, anaerobic effluent) – (ammonium, anaerobic process effluent mass loading) (7.1 g/g ammonium removed) = (14,450 g/d) + (1859 g/d) (7.1 g/g) = 1251 g/d

4. Estimate effluent constituent concentrations for conservative constituents using a mass balance approach. Because the flow will be equalized fully, use the mean mass loading. Because these constituents are conservative in the treatment system, the daily mass loading is equal to the mass in the excess flow that goes to dispersal. Therefore, the recycled water concentration is equal to the mass loading divided by the flow to the dispersal system. The flow to the dispersal

system is equal to the difference between the total wastewater flow and the recycled water for toilet flushing (1845 gal/d – 1582 gal/d = 263 gal/d).

Constituent	Influent mass loading, g/d	Process effluent mass loading, g/d		Effluent concentration, mg/L
		Anaerobic	Aerobic	
TDS	7593	7593	7593	7618[a]
TP	157	157	157	158

a. TDS, effluent = (TDS influent mass loading) (conversion of g to mg) / [(flow to dispersal system) (conversion of gal to L)] = (7593 g/d) (1000 mg/g) / [(263 gal/d) (3.785 L/gal)] = 7618 mg/L.

5. Determine preliminary process size, based on mean loading criteria from Table 4-2.

   a. SPT volume estimate

      Volume of SPT (minimum) = (future mean flow) (factor to account for volume for solids storage) = (2399 gal/d) (1.3) = 3119 gal

      It is recommended to use the next largest commercially available tank for additional solids storage capacity and other unexpected events (e.g., an open flush valve). In this example, the designer may consider a 3500 or 4000 gal tank.

   b. USCT sizing

      Area of single USCT = (peak future flow) (m^3 per gal) / [(period of peak flow) (maximum upflow velocity)] = (4798 gal/d) (0.003785 m^3/gal) / [(10 h/d) (1 m/h)] = 1.82 m^2

      Volume of single USCT = (liquid depth) (minimum area) = (2 m) (1.82 m^2) = 3.6 m^3 = 960 gal

      Use two 1000-gal (minimum) USCTs in series.

   c. UAF sizing

      Area of single UAF = (peak future flow) (m^3 per gal) / [(period of peak flow) (maximum upflow velocity) (fractional packing void space)] = (4798 gal/d) (0.003785 m^3/gal) / [(10 h/d) (1 m/h) (0.95)] = 1.91 m^2

      Volume of UAF = (liquid depth) (minimum area) = (2 m) (1.9 m^2) = 3.8 m^3 = 1010 gal

Use two 1000-gal (minimum) UAFs in series.

d. BTPT volume estimate

Volume of BWFDT = future summer flowrate = 2783 gal/d = 3000 gal

Volume of BWFET = 1500 gal/d

Volume of SSFDT = 2000 gal/d

Use a three-compartment 6500-gal tank (i.e., 3000 + 1500 + 2000)

e. BWF preliminary design

Number of BWFs = (ammonium nitrogen mass loading) / (conservative filter nitrification capacity) = (1859 g N/d) / (650 g/d) = 2.9

Note that nitrification rates of up to 950 g/d were observed in BWF units following installation of low flow fixtures and nitrate addition to the primary tank.

Use three 4 ft x 4 ft filters; additional filters can be added later if needed.

f. SSF preliminary design

Area of SSF = (future mean flow) / (hydraulic loading rate) = (2399 gal/d) / (40 gal/ft^2•d) = 60 ft^2

Area of one SSF with 6-ft diameter = 28 ft^2

Use three 6-ft-diameter tanks; one filter can be taken out of service for cleaning.

g. OCT and ozone process sizing

Ozone demand = (Venturi flow) (L per gal) (ozone dosage) (conversion to g) (conversion to h) = (10 gal/min) (3.785 L/gal) (50 mg/L) (0.001 mg/g) (60 min/h)= 114 g/h

Operational time = [(minutes per day) (flowrate through Venturi)] / (future summer flow) = (2782 gal/d) / (10 gal/min) = 278 min/d

Ozone output required at 75 percent transfer efficiency (maintain efficiency through maintenance or replacement)

Ozone dose = (ozone demand) / (transfer efficiency) = (114 g/h) / (0.75) = 151 g/h

Volume of OCT = (flowrate)(hydraulic retention time) = (10 gal/min) (1 min) = 10 gal

Use a single 10 gal contact tank, 10 gal/min venture injector, and two 150 g/h ozone generators operating in parallel. Note that the proposed ozone generator strategy is designed for two generators to run at partial capacity in parallel and will also allow the system to be run with one generator out of service.

h. CCT and chlorine dosing system sizing

Volume of CCT = (future summer flow) (hydraulic retention time) / (hour per day) = (2783 gal/d) (3 h) / (24 h/d) = 348 gal

Chlorine = [(assumed chlorine dose) (future mean flow)(L per gal)] / (chlorine concentration) = [(15 mg/L) (2399 gal/d) (3.785 L/ gal)] / (100,000 mg/L solution) = 1.36 liters per day (L/d) (Note: 100,000 mg/L is equal to a 10% solution)

Use three 150-gal (minimum) tanks-in-series. The tanks-in-series model was piloted at the Dunnigan SRRA to approximate a plug flow chlorine contact tank; additional measures, such as packing and baffling, are recommended to further improve hydraulics.

The chlorine storage tank can be sized for monthly refill at (30 d) (1.36 L/d) = 41 L

i. PWST sizing

Volume of PWST = future summer flowrate = 2782 gal/d = 3000 gal (rounded to nearest commercially available size)

j PT sizing

Volume of PT = 1000 gal

Comment

The preliminary process sizing shown in this example is based on typical design values. Final process designs will be based on actual site conditions, including temperature and diurnal variations. Procedures for developing a final process design are described in the following design sections. Note that tanks are commercially available only in certain sizes. Calculations should be used as a guideline to select the best size option available. Furthermore, calculations are performed to select individual tanks to be used in series. For example, Step b is for sizing a tank based on upflow velocity to flow through one tank, and then the next. The USCT and UAF are generally the same size, but may differ based on void space.

4.3 ANCILLARY REQUIREMENTS

Equipment installed at SRRA facilities should be protected from the environment and vandals. Considerations include roof coverage, equipment storage, secure areas and perimeter fences and walls, alarms, and video surveillance. In cold weather areas, it may be necessary to bury pipes or protect on- or above-grade pipes and tanks from freezing using insulation and/or a tank heater, heat tracing for pipes, and room heaters. To best represent the system at that point, no special treatment should occur when collecting samples. These topics and odor control systems are discussed in this Sec. 4.3.

4.3.1 Environmental Protection

It is recommended that the all PVC piping and chemical and biological process tanks be placed under a roof or canopy to protect them from solar radiation (UV radiation), heat, and rainfall. Tanks that should be sheltered are the BWFs, the SSFs, the CCT, and possibly the PWST. The biological processes will benefit from the more stable conditions present under a shelter. In areas with rain and snow, the shelter will also protect workers who maintain the equipment.

Sensitive electronic equipment should be kept in a protected and air-conditioned space: the oxygen concentrator, ozone generator, online water quality sensors, field instruments, and control panels. A small insulated building equipped with an HVAC system should be sufficient protection from dust, moisture, and temperature extremes; this will greatly extend the life and reduce the maintenance required to keep the water system in operation. In addition, operation of the system will be less burdensome for maintenance staff because the operations will take place in a sheltered location.

Freezing conditions are not a concern for buried pipes in California, but could freeze water inside exposed piping. It is common to bury or insulate exposed pipes to protect them where freezing conditions are expected. The equipment shelter described above will likely be sufficient to prevent freezing of water in pipes in mild weather areas. Cold weather also degrades the performance of biological processes because the metabolic rate of treatment microorganisms is related to temperature. To allow proper operation in cold climates, submersible tank heaters and water heaters have been used. Cost and feasibility analyses should be considered prior to implementing an OWTS reuse system, where weather conditions could inhibit performance. Temperature effects on biological processes are described in Sec. 3.4. Other site considerations are described in the following sections.

4.3.2 Site Security

It has been observed that vandalism and theft are more of a concern at some SRRA facilities than others. Examples of vandalism are broken porcelain fixtures, indoor and outdoor graffiti, and fires. Items typically stolen include light fixtures, wire, data collection devices, trash cans, and brass valves. Therefore, it is important to take measures to minimize theft and vandalism so that water systems remain functional. Examples of measures are the installation of fencing, security cameras, motion-detecting light systems, alarms, and frequent visits from California Highway Patrol (CHP) staff.

4.3.3 Odor Control

Typically, odors result from problems with venting of lift stations and anaerobic tanks. Another location for potential odors is where wastewater is pumped onto the BWF. Lift stations should be connected to the SRRA comfort station vent stack so that odors are dissipated above ground level. Remote tanks should have independent vent systems that promote the movement of air through carbon filters or biofilters. Standpipes or inline fans can be used to generate draft for odor control.

4.4 SYSTEM LAYOUT

The specific layout of a water reuse system depends primarily on the site constraints. Because most of the processes are modular and there are a number of pumps involved, the location of process tanks and equipment is somewhat flexible. In general, it will be advantageous to locate the process equipment in one area for improved maintenance. Layout considerations for process components are summarized in Table 4-3. A chart for sizing fiberglass tanks is shown on Fig. 4-2. An example plan layout for the system developed in Ex. 3 is shown on Fig. 4-3.

Table 4-3
Summary of layout considerations for key process components

Element	Layout considerations
LS	Lift stations (LSs) should be placed close to the comfort station as needed to maintain adequate slope (e.g., 2 percent) and minimize drainage pipe length for water-conserving plumbing fixtures. Because lift stations can be made relatively deep, there is some flexibility in locating these structures. Pumps and internal components will need to be removed periodically for maintenance. Access should be provided for vacuum trucks for periodic solids removal or pumping contents for maintenance activities.
SPT USCT UAF	SPTs are to be placed in series and adjacent to the USCT, UAF, and BTPT if gravity operation is to be used. All of the tanks are recommended for below-grade installation so that gravity transfer can be used and to retain thermal energy. If it is not possible to co-locate these tanks, pumps can be used to transfer flow through sequential processes. Removal of solids will be needed periodically, so an access road should be provided to facilitate tank maintenance activities. The solids removal for the SPT is more involved and frequent than for the USCT and UAF. The UAF will additionally need access for placement and cleaning of the packing. Ideally, these tanks will be placed away from areas accessible by the public.
BTPT	BTPTs are placed below grade and must be adjacent to the backwash filter for proper operation. If the BTPT cannot be located near the SPT, a transfer pump may be needed to deliver flow. Pumping of the BTPT may be needed infrequently, so vehicle access will be needed.
BWF	BWFs are put in place on-grade using a forklift. Because the influent and effluent from the BWF are stored in the BTPT, it is important to locate these process tanks in the same area to minimize excavation and to improve operation. It is recommended to install under roof or canopy.
SSF	The head from SSFs located on-grade can be drained back to the SSFDT at the time of cleaning to maximize water recovery, but also requires that the BTPT be located in the same general area as the SSF. Maintenance of the SSF requires removal of small amounts of sand; therefore, access will be needed to climb in and out of the filter. Vehicle access is needed for delivery and removal of sand media. It is recommended to install the filter under a roof or canopy.
ODT OCT	Ozone dosing tanks (ODTs) and ozone contact tanks (OCTs) need to be located in or near the instrumentation and control building (ICB). Effluent from the SSF is pumped to the ozone Venturi injector and therefore can be located remotely from the BTPT. Limited vehicle access for delivery of light equipment and bottled chlorine is sufficient, as no heavy loads or maintenance trucks are expected at the ICB. It is recommended to install under canopy.
CCT	CCTs can be located where convenient but typically near the ICB to minimize pipe runs. It is recommended to install the tanks under a roof or canopy.
ICB	The instrumentation and control building is a small temperature-controlled building for housing sensitive electronic equipment, including sensors, SCADA, and equipment.
PWST	PWSTs are best located below grade for gravity operation or where the PWST volume is less than about 20,000 gal, and on-grade for larger tanks with pumped effluent transfer from the CCT. Booster pumps are used to transfer product water from the PWST to the PT.
PT	PTs can be located in a secure area in almost any location because booster pumps will be used to pressurize the flow. Pressure drops should be considered when the PT is located far from the comfort station.

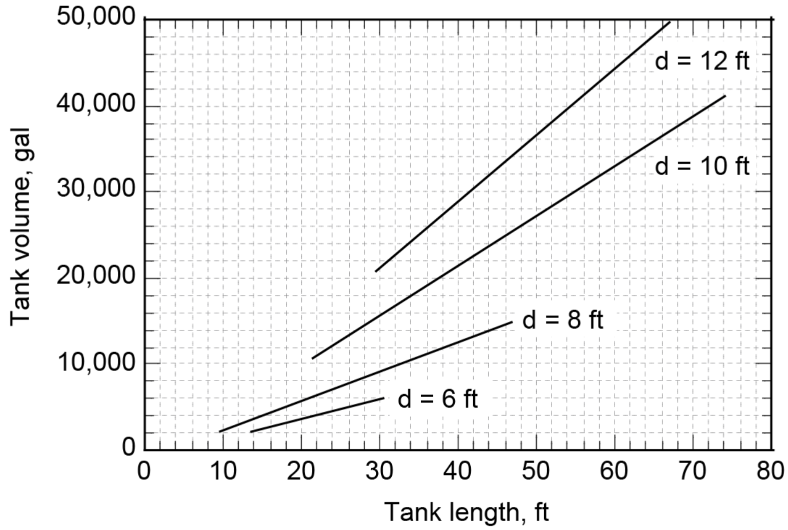

Figure 4-2
Fiberglass tank sizing chart for diameters ranging from 6 to 12 ft

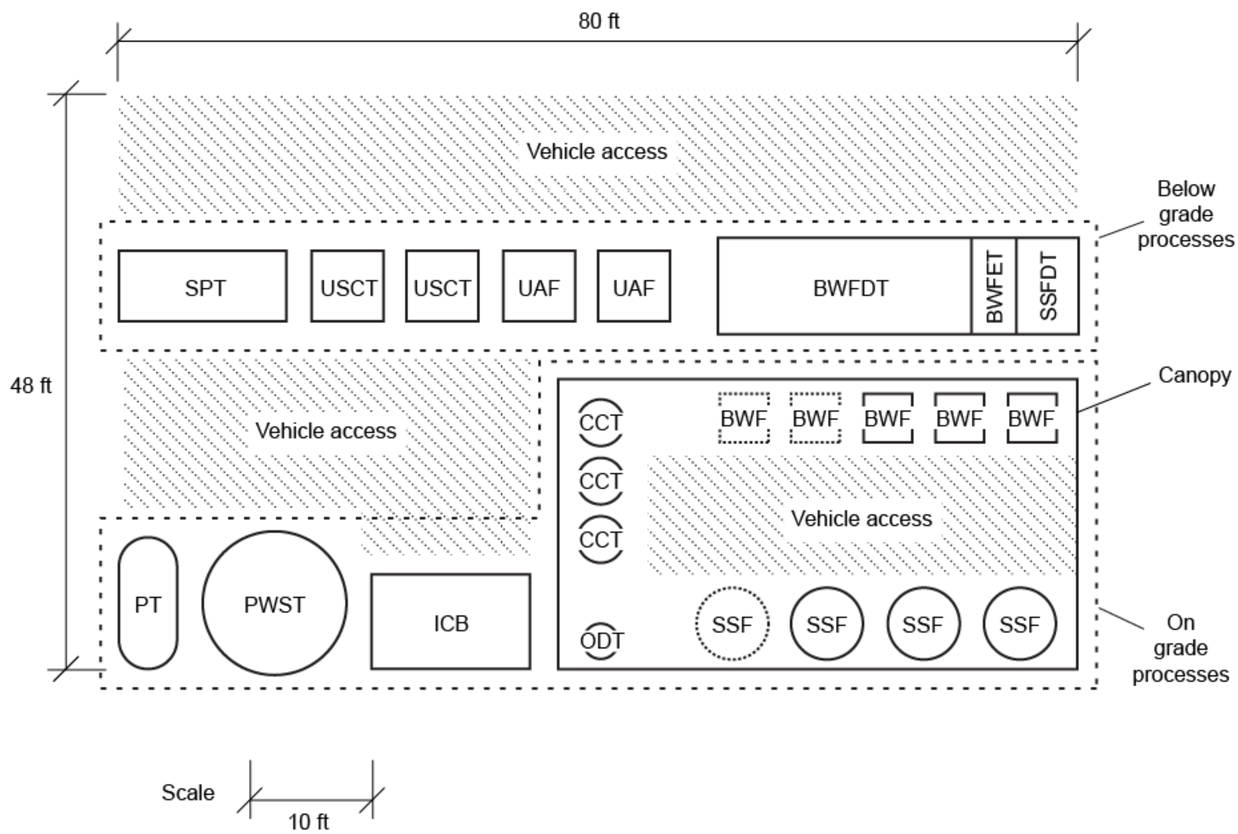

Figure 4-3
Layout for system developed in Ex. 3. See Table 4-3
for process terminology and siting considerations.

As shown on Fig. 4-3, an overall area of about 4000 square feet (ft^2) will be needed to accommodate the entire treatment process. The recommended layout maximizes the gravity transfer of water from one process to the next. Unit processes that benefit from enhanced inspection, maintenance, and environmental control are placed inside of a building or under a canopy and located on-grade. A perimeter fence and access for fire or emergency equipment should be considered on a site-specific basis.

5.0 WASTEWATER COLLECTION

Wastewater from comfort stations typically contains a greater concentration of trash and debris than is observed in conventional sewer systems. These gross solids range from personal care and healthcare products to diapers and clothing. The wastewater collection system needs to operate under these extreme conditions and reliably deliver this wastewater to the downstream treatment processes. Low-flow fixtures and waterless urinals paired with the high level of gross solids make the wastewater collection a unique challenge.

This section describes the approach for developing a wastewater collection system design, as shown in Chart 5-1.

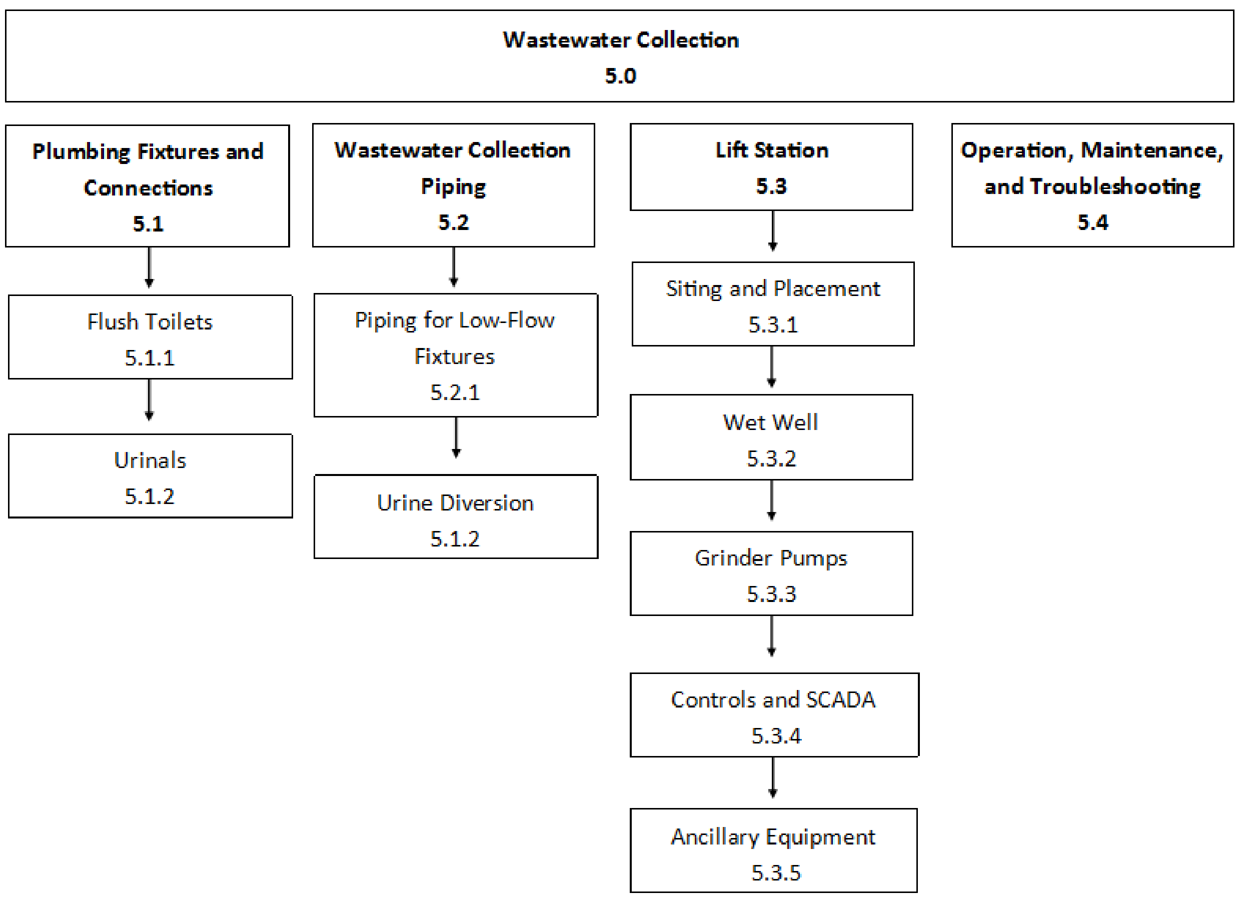

Chart 5-1
Wastewater collection system design

5.1 PLUMBING FIXTURES AND CONNECTIONS

Plumbing fixtures in the comfort stations are the beginning of the treatment system. Their use and functionality can have significant impacts on the downstream treatment components. In rest areas undergoing renovation or new construction, plumbing fixtures are being retrofit with 1.28 gal/flush toilets and 0.125 gal/flush (1 pint) urinal fixtures as required in Green Building Code and California Governor's Executive Order B-29-15. Design and maintenance considerations are summarized below.

5.1.1 Flush Toilets

Flush toilets are responsible for almost all the bulk solids introduced into the wastewater system. These solids are the primary cause of blockages in the wastewater conveyance systems. Low-flow fixtures, with their reduced flow per flush, are considered a potential cause of blockages in the conveyance system. An example of trash causing blockages is shown on Fig. 5-1.

(a) (b) (c)

Figure 5-1
Views of SRRA drainage system: (a) scale accumulation at pipe opening, (b) clog that occurred at pipe outlet two days after the switch to low-flow toilets, and (c) trash removal from lift station.

Drainage Transport of Solid Waste and Buildings (November 2012) published by the Plumbing Efficiency Research Coalition (PERC, 2012) showed that significant variables in transporting waste are flush volume, toilet paper selection, and pipe slope. The study also found an orderly and predictable movement of solids in the test apparatus. The test apparatus was constructed of new 4-inch polyvinyl chloride (PVC) pipe, at 1 and 2 percent slope.

For low-flow flush toilets to be effective in reducing water consumption, the flush valves must be in good working condition. The flush valves need to be specified with chemical-resistant diaphragms to withstand the effect of chloramines found in many SRRA water supplies and that will be present in recycled water generated at the SRRAs.

Flush valves are known to wear out over time and need to be replaced approximately annually according to manufacturer specifications. Water with fine grit or aggressive constituents may require more frequent maintenance. Fixture flowrates should be checked periodically to confirm proper operation of the flush valve and to identify leaks. In modeling fixture flowrates, a factor for valve fouling should be included to account for wear of the flush valve diaphragms over time, depending on the level of maintenance (as discussed in Sec. 3.2.2).

Selection of an off-white fixture color also will have a positive impact on water use, particularly with recycled water. By reducing the contrast between the recycled water and the bowl color, excess flushing and aesthetic issues are expected to be minimized. A view comparing groundwater with a value of 0 platinum-cobalt (pt-co) color units (cu) and recycled water at 40 cu is shown on Fig. 5-2. A color value between 5 and 40 cu was readily achievable at the Dunnigan SRRA pilot facility and was a key criterion for ensuring process performance.

Groundwater Recycled water (color = 40 cu)

(a) (b)

Figure 5-2
Views of biscuit toilet installed at Dunnigan-NB SRRA:
(a) with groundwater in bowl, and (b) with recycled water with a color of about 40 cu.

5.1.2 Urinals

In a 2008 report to Massachusetts Executive Office of Energy and Environmental Affairs and the Executive Office of Administrative and Finance, Operational Services Division prepared by Industrial Economics, Incorporated and Aceti Associates (IeC, 2008) conducted interviews with facility managers and custodians that maintain waterless urinals at ten facilities, including offices, educational buildings, gyms, dorms, and one prison. Seven of the ten facilities reported an overall positive experience with waterless urinals; two facilities reported an overall negative experience, and one facility had a mixed experience. Facilities that installed waterless urinals as part of new construction had uniformly positive overall outcomes, reporting only isolated problems. In contrast, retrofit projects posed additional challenges that need to be addressed.

A facility maintenance protocol and the type of urinal cartridge used appear to be related to implementation experience at some facilities. The two facilities that have had negative experiences with waterless urinals were a dormitory and a prison. In addition to the retrofit issues discussed above, it appears that problems experienced at these facilities are exacerbated by (1) the relatively high, constant use that these urinals receive, and (2) the potential for misuse and unmet needs for more frequent cartridge changes. Three potential factors studied do not appear to influence implementation experience: lack of maintenance staff training (all facilities report adequate training); mixed installation (i.e., conventional and waterless installed urinals in the same building); and age of the urinal (facilities did not report an increase in problems with older waterless urinals.) (IeC, 2008)

The following recommendations, which are appropriate for Caltrans and its comfort station retrofit and upgrades, were presented in the report:

- Waterless urinals appear to work well in most settings.
- Installation of waterless urinals is more straightforward during new construction; retrofitting them in existing bathrooms without renovation presents some challenges, but can be accomplished successfully. Prior to a retrofit project, it is imperative that facilities (1) ensure that the slope of the drain line is at least 2 percent, (2) route drain lines to avoid problems such as sediment buildup, and (3) check that drain heights are appropriate to the brand to be purchased. Facilities are far less likely to encounter problems with retrofit projects if these preparations are made.
- Availability of maintenance staff is important for both daily cleaning and frequent cartridge changes/refills at facilities with high user loads.
- Checking and changing waterless urinals cartridges often leads to escape of ammonia odors in the bathroom, causing a temporary but significant odor problem. The O&M manual does not require any specific safety procedures. However, to avoid the unpleasantness of odor escape during cartridge changes,

a cup of vinegar can be added to the trap during cleaning or staff can wear a mask that reduces odors.

There is a general concern of sediment buildup from waterless urinals. Urine, while 96 percent water, contains small amounts of dissolved solids. Some users have expressed concern about buildup of urine sediments in drain pipes when using waterless urinals. However, studies on the effects of urine on drain pipes have shown that buildup is primarily the result of a reaction between the mineral content of water and chemical transformations that occur when urea hydrolyzes in the trap. Waterless urinal manufacturers recommend flushing drain lines monthly to keep them clear; other users have reported no buildup problems. Each drain trap design and its corresponding maintenance protocol have features that seek to prevent sediment buildup. To flush down any soft sediments that have accumulated in the drainpipe between cartridge or insert replacements, maintenance protocols generally call for 2 to 5 gal of water to be poured down the drain after the cartridge trap or insert is removed for replacement (IeC, 2008).

In new construction, the placement of urinals at the end of the drainage line rather than at the beginning may help with the potential impacts of corrosion and mineral buildup.

5.2 WASTEWATER COLLECTION PIPING

As water use reduction and reuse become more prevalent, considerations with respect to the wastewater collection systems from the comfort station become more important. Many existing wastewater collection pipe systems have been designed assuming relatively high-flow fixtures and proportionately low solids content.

5.2.1 Piping for Low-Flow Fixtures

While lower flow fixtures are beneficial for reducing flows from the comfort stations, they have the potential to impact the performance of the wastewater collection systems, particularly in retrofits. With new plastic pipes with slopes between 1 and 2 percent, no additional clogging should occur as a result of new low-flow fixtures. The 1.28-gal (4.8-liter [L]) and 1.6-gal (6.0-L) test runs by PERC TC resulted in orderly and predictable movement. As a result, PERC TC anticipates no problems with use of 1.28-gal/flush (4.8-L/flush) high-efficiency toilets (HETs) in new commercial construction (PERC, 2012).

In retrofit applications, low-flow fixtures may have the potential to increase blockages with the flat slopes and aged cast iron pipes with potential mineral buildup and/or corrosion that could be found in existing wastewater collection infrastructure. In retrofit applications, wastewater collection piping should first be inspected for defects, root intrusions, sagging, or other physical conditions that could result in clogging with lower flush volumes (PERC, 2012).

5.2.2 Urine Diversion

Diverting urine from the urinals may be necessary as a technique to meet regulatory requirements for nitrogen reduction or may be desirable in the future for nutrient recovery. The most effective way to divert urine at most comfort stations is by collecting flow from urinals independent of the toilet and sink flows. The diverted urine can be sent to a holding tank for sequestration or removal, evaporation ponds for dehydration, or to an onsite nutrient recovery process.

Sequestration would be used where a nitrogen limit is mandated and the treatment system is unable to meet the nitrogen concentration standard during that time period. A likely scenario would be during a period with extended lower temperatures that may impact the biology responsible for nitrogen conversion. The urine could be stored until the treatment system is robust enough to treat the nitrogen. At that time, the urine would be metered back into the system via the pump installed in the urine tanks. Urine would be pumped to the lift station and blended with the incoming wastewater from the toilets and sinks. Removal would require pumping of the stored urine in the urine tank. If the treatment system is unable to process all the urine, it can be pumped and hauled to an appropriate facility. Several research studies are investigating the direct application of urine to soil as a fertilizer. In general, there are concerns about the impact of salts on plant growth and the fate of pharmaceuticals and other chemicals present in urine. An alternative process known as CrystalStrip (see Fig. 5-3) has been developed as a fully automated system to extract magnesium ammonium phosphate and ammonium bicarbonate from urine. In the future, this type of technology could have a beneficial impact on the design and operation of onsite water recycle systems.

Figure 5-3
Automated CrystalStrip process to remove nitrogen and phosphorus from urine to produce concentrated fertilizer products (Leverenz and Adams, 2015)

5.3 LIFT STATION

A lift station consists of a wet well, grinder pumps, controls, and ancillary components such as valves, etc. The basic purpose of the lift station is to transfer wastewater from the source to a point of treatment, provide a small volume of storage between pump cycles, and provide some emergency storage to allow for a response to catastrophic failures, pump failure, power outages, etc. However, the lift station configuration has a major impact on the ability of the pumps to convey the solids introduced to the lift station. With the high-bulk solids loading seen from the comfort stations, the design of the lift station can have a significant impact on the performance and reliability of the entire wastewater treatment system.

Modern lift stations are most commonly designed using a cylindrical vessel made of either concrete or fiberglass. In some situations, rectangular lift stations have been used, especially in high-flow applications where large pumps are required and a cylindrical tank is not practical to house the pumps due to their size. Given the relatively small flows from comfort stations, the focus of this section is on cylindrical lift stations. The use of fiberglass lift stations has eliminated the adverse impacts of corrosion and reduced the cost of installation. Access covers and components should be constructed from aluminum to reduce corrosion from hydrogen sulfide gas and to improve accessibility with lightweight materials.

Lift stations can be designed using the individual components or more commonly as an integrated package from a manufacturer. The basic components of a lift station and the design points to consider when designing lift stations to serve comfort stations are outlined in the following section. The basic lift station design used at the Dunnigan-NB SRRA pilot facility is shown on Fig. 5-4.

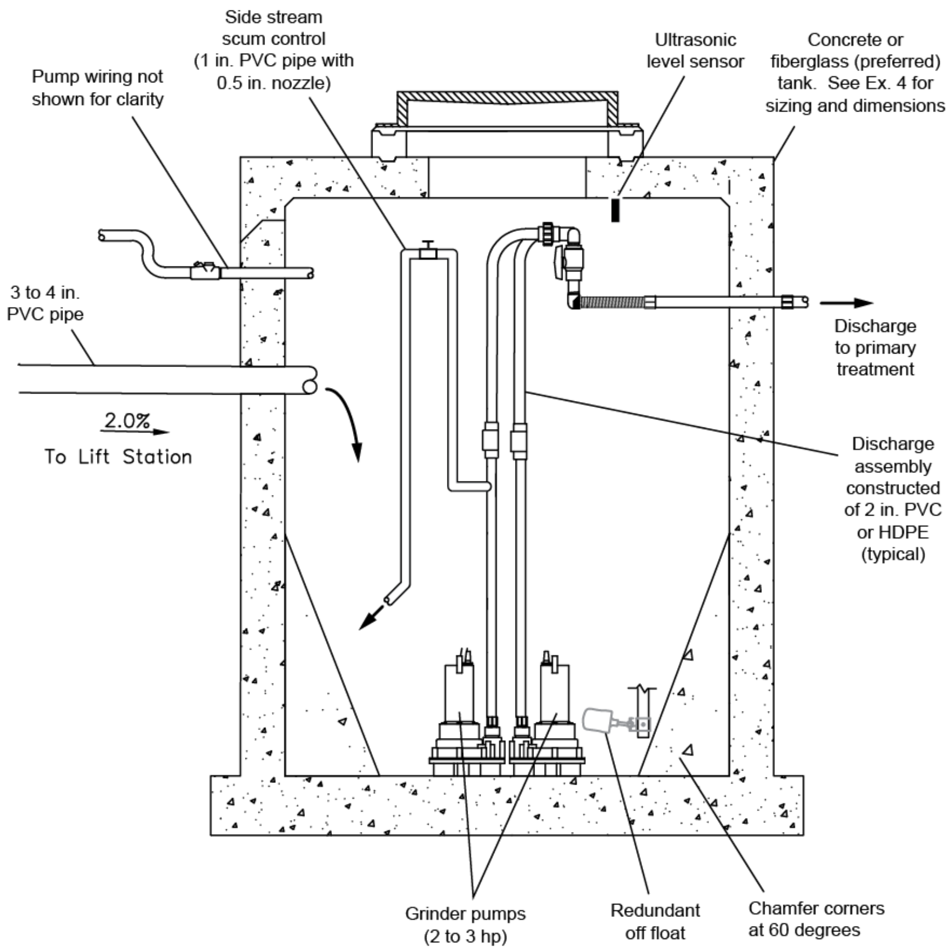

Figure 5-4
Lift station schematic for Dunnigan SRRA pilot facility

5.3.1 Siting and Placement

Under ideal circumstances, the lift station will be placed in an area that does not impact monitoring and maintenance activities. Generally, the lift station should be installed as close as practicable to the comfort stations directly in line with the discharge plumbing from the fixtures in the comfort station, minimizing turning structures and other potential points for clogging to occur.

Consideration should be given to monitoring and maintenance activities when selecting a suitable location. Most lift stations will have large pumps requiring a small crane or A-frame to remove the pumps for servicing. The path of travel for service vehicles and personnel must be considered as part of the siting. Overhead conflicts with lighting, trees, and other structures must also be considered. Maintenance and monitoring of the lift station is facilitated by an access hatch at the top of the lift station. Shallow lift stations facilitate monitoring activities such as visual inspection of solids, verification that level sensors are free of debris, and sample collection. The solids removal process at the Dunnigan-NB SRRA pilot facility lift stations is shown on Fig. 5-5.

Figure 5-5
Removing solids from a lift station for pump replacement

Lift station maintenance, including removal of accumulated solids, is also accomplished more effectively when the minimum liquid level is closer to the ground surface. There should be sufficient space around the lift station such that it can be accessed for cleaning and maintenance. In addition to space, a hose bib that can provide water at 20 gal/min and a pressure of 50 pounds per square inch (lb/in^2) should be provided to aid in maintenance activities. Traffic loading and setback distances should follow manufacturer and building code specifications.

5.3.2 Wet Well

The plumbing entering the wet well is an important design detail. An inlet too high can lead to odor generation and entrainment of air in the wastewater from the high velocity of the wastewater entering the lift station. Excess air in the wastewater can impact the performance of the pumps and may lead to early pump wear. This concern needs to be balanced with the value a higher than normal inlet height can have on floatable solids commonly found in comfort station wastewater.

The lift station at the Dunnigan SRRA was tested with a sanitary tee installed to provide an inlet within 2 ft of the bottom of the lift station. In this quiescent environment typical of conventional lift stations, the floatable solids formed a solid mass that held back additional solids. The pumps drew the liquid out of the solids, exacerbating the problem. The result was a dense mass of solids that could make their way to the grinder pump. The sanitary tee was removed to encourage agitation in the wet well.

If odor generation or air entrainments are a major concern, a sanitary tee should be used at the inlet to direct flow away from the inlet of the pumps and should be kept to less than 3 to 4 ft of fall. Hydrogen sulfide gas has also been found to corrode uncoated concrete and galvanized steel access doors (Fig. 5-6).

The wet well configuration is important for the function of the lift station. The shape of the wet well floor works in concert with the pumps to deliver solids to the downstream treatment components. Proper design of the wet well prevents bottom sedimentation and helps prevent scum formation on the surface. To optimize performance, the wet well should have angled walls at a minimum angle of 45 degrees and up to 60 degrees, if constructible.

Figure 5-6
Corrosion of galvanized metal access door to lift station at Willows SRRA

5.3.3 Grinder Pumps

Grinder pumps are the preferred method for conveying comfort station wastewater. Grinder pumps are selected over solids handling pumps because of their ability to macerate the bulk solids and convey those solids to the primary treatment process. Many of the gross solids found in the wastewater at comfort stations are larger than the nominal 2- to 3-inch-diameter solid that most small solids handling pumps are designed to pass. The use of grinder pumps allows for smaller force main sizing, and reduces clogging and gross solids accumulation in the lift station, which would require pumping and disposal.

A number of pump manufacturers supply grinder pumps for applications similar to the SRRA comfort stations. The lift station at the Dunnigan SRRA facility was tested with a small, relatively low-cost Liberty LSG200-Series Omnivore® 2-horsepower (hp) submersible grinder pump, 230-volt (V) single phase. These pumps experienced a few different modes of failure, including internal capacitor failure and rags binding on the cutter.

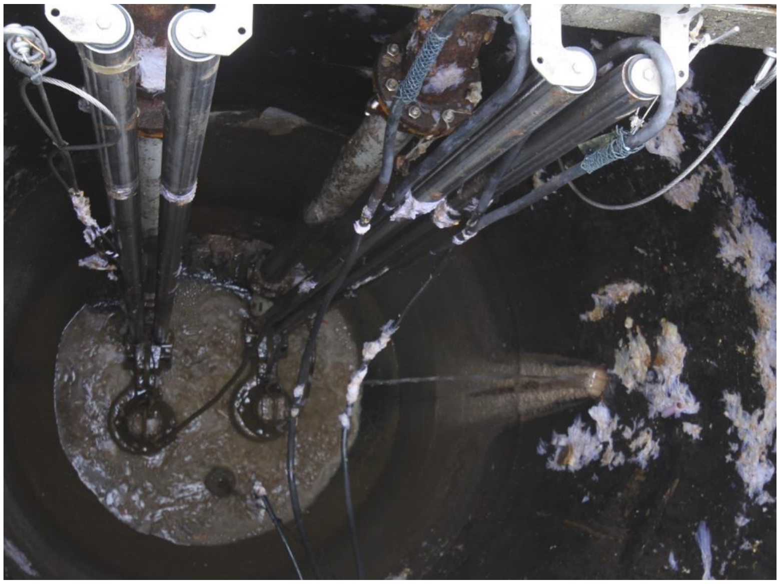

Figure 5-7
Views of grinder pumps: (a) 2-hp Liberty grinder with rags around the cutter at the Dunnigan SRRA, and (b) rail-mounted 3-hp Flygt grinder with pump down to intake to clear solids at the Maxwell SRRA

Grinder pumps are selected using standard engineering practice, considering static head elevations between the water surface in the lift station and the discharge point at the downstream treatment system. Friction head can be calculated using the Hazen-William equation for head loss due to friction. In most SRRA applications, the total head loss is less than 60 ft at flowrates between 30 and 50 gal/min. In most applications, grinder pumps for SRRA applications will be in the 2- to 3-hp range (3 hp is generally preferred).

Discharge outlets from grinder pumps in the 2- to 3-hp range from 1.25 to 2 inches. Discharge plumbing will include an in-line check valve and/or anti-siphon valve along with shut-off valves for maintenance activities.

The size of the force main to the downstream treatment processes will be determined on a case-by-case basis. With anticipated flows from the comfort stations and assumed pump delivery rates from the grinder pumps at less than 50 gal/min, the force main sizing will be between 2 and 3 inches. Force main material can be ductile iron, solvent welded PVC (SDR 21) or fused high-density polyethylene (HDPE); material selection will be based on site conditions and method of installation, either open trench or directional drilling.

The grinder pump power supply can range from 230V single phase to 460V three phase depending on the power service available at the SRRA. Three phase pumps will run at lower amperages and are more efficient than their single phase counterparts. Grinder pumps should be installed in pairs in a lead/lag configuration where the pumps are alternated each cycle. In the event of high flows or a failed pump, the lag pump will be engaged. Grinder pumps should be installed on a guide rail system allowing for simple removal and replacement of the grinder pumps during maintenance activities. Care should be taken when designing a lift station with basic components. For example, it is important to ensure that a guide rail system and its pump spacing and tolerances will fit through the access opening specified and sit in the flat basin below the sloped walls of the lift station (see Fig. 5-4). With the high solids found at SRRA comfort stations, the lift station will require full pump downs periodically or other considerations to keep the solids levels to a manageable level. It is recommended to keep a spare onsite in case of pump failure.

Discharge plumbing can be designed to return a portion of the pump discharge to the wet well and agitate the solids so they do not become a large floating mass that cannot be pumped out by the grinder pumps. For example, as shown on Fig. 5-4, a side-stream scum control system was developed at the Dunnigan SRRA pilot facility to reduce solids accumulation in the lift station. Some manufacturers can include this is as part of their pump heads.

5.3.4 Controls and SCADA

The purpose of the lift station control system is to provide logic to energize and de-energize the pumps as needed to move the liquid and solid constituents to subsequent processing. In some cases, the control system is also used to run pump down cycles or other operations used to remove accumulated floating solids from the lift station. The SCADA system is used to monitor and control the lift station remotely, as described below.

The basic input signal to the control system is the liquid level in the wet well. Floats have been the standard in the wastewater industry for decades and are the most basic way to convey lift station levels to the control panel and SCADA system. However, as observed at the Dunnigan SRRA, floats can be weighted down with solids entering the lift station, impacting their operation and effectiveness. The result is increased maintenance frequencies and exposure of maintenance personnel to contact with the lift station contents. A description of various liquid level sensing devices is in Table 5-1.

The controls for the lift station should be placed within line of sight to the lift station ease monitoring and maintenance and provide a higher level of safety during routine maintenance activities. Controls can be placed in a cabinet adjacent to the lift station or mounted to a nearby structure such that the lift station can be observed while adjusting the controls. The controls cabinet must be rated National Electrical Manufacturers Association (NEMA) 4X and must be constructed to be tamperproof and resistant to vandalism.

Controls will require current sensors motor protection for all the pumps. Pump amperage can be logged and displayed as part of the SCADA system. Changes in pump amperage can be an indication that the pumps or lift station components need service. Parameters that are monitored and/or controlled via online instrumentation and a remote SCADA system are summarized in Table 5-2.

5.3.5 Ancillary Equipment

Some type of ventilation system should be provided to control the accumulation of hazardous and odorous gases. When the lift station is connected directly to the building plumbing, it is possible to use the building vent stack to disperse these gases into the atmosphere.

Buoyancy of the lift station should be considered in the design process to ensure that the lift station does not come out of the ground in areas with high seasonal or permanent water tables. Anti-buoyancy ties downs and straps may be required to keep the lift station in the ground.

Table 5-1
Summary of alternative water level sensing equipment

Sensor	Description
Floats	A float control system uses a series of signal float switches installed in the lift station and relays the level information to a programmable logic controller (PLC). Floats are typically an inexpensive option for level sensing, but do not provide actual level sensing. Floats do not work well in environments where their operation can be impeded by floating solids.
Bubbler systems	Bubbler systems consist of a tube installed in the lift station, an air supply compressor, and a pressure switch or pressure transmitter. The liquid level in a station can be equated to the hydrostatic pressure measured at the bottom of the well. To measure the pressure, a constant air volume can be pumped through a tube that exits at the bottom of the lift station. By measuring the pressure inside the tube, the level in the lift station can be determined. These systems are relatively low cost and can be used for level sensing as well as set point control of pumps. These systems require a mechanical air supply that requires maintenance and the bubbler outlet will require periodic inspection and cleaning
Ultrasonic transmitters	Ultrasonic level sensors provide a continuous analog output by sending a sound pulse down to the surface of the sewage in the lift station. The sensor reads the return signal and the controller calculates the sewage surface distance from the sensor. Ultrasonic transmitters are a non-contact measurement, so the sensor is mounted at the top of the well. These systems will typically consist of an analog sensor in the well that signals a standalone control or PLC-style controller to initiate the pump starts and stops. This information can then be fed into the SCADA system. Ultrasonic level sensors are a more expensive option and require consideration in the design and installation to ensure there are no obstructions (i.e., pipes, cables, rails, pumps, etc.) that would impede the ultrasonic pulse. This is an important consideration when sizing the wet wells with the pup rails, basin walls, and inlet and outlet piping. Ultrasonic sensors need to be calibrated at startup. Because these sensors are not designed for submerged applications, protection of the transmitting head from inundation is essential. The transmitting head should be inspected after any high level alarm condition to verify that the sensor has not been damaged and is functional.
Submersible level transmitters	Similar to a bubbler, submersible level transducers measure the hydrostatic pressure from the bottom of the lift station. The sensor is placed at the bottom of the lift station and the transducer provides continuous analog signal output (4-20 milliamps [mA]) relative to a fixed level range. This information can then be fed into the SCADA system. The sensor requires an atmospheric reference pressure for calibration, which is usually provided via a small-diameter tube to the open air space in or outside the lift station. Submersibles can be a cost-effective method for continuous level monitoring and pump controls. Periodic cleaning of the senor may be required as well as inspection of the reference pressure tube; the reference pressure tube must not become submerged or filled with water or condensate. The tube can be kept dry by lacing the reference pressure tube outside the lift station.

Table 5-2
Recommended parameters and control descriptions

Parameter	Description
Liquid level	Liquid level is monitored via an ultrasonic water level sensor in the lift station. This liquid level is used to control the on and off set points for the pumps and to provide local and remote alarms via the SCADA system. The lift station is designed typically as an on-demand system, unless it is to also be used for flow equalization.
Low level alarm (LLA)	The LLA is also known as the redundant off (RO) sensor that is used to detect low levels in the lift station and to turn off the pumps. This control protects pumps from running in a dry condition.
Pump off set point (POFF)	The POFF is used to turn the pumps off at a designated level, typically the pump's minimum liquid level.
Pump on set point (PON)	The PON is used to turn the lead pump on at a designated level. This level is set on the basis of the following variables: inflow, downstream process flow acceptance, pump duty cycles, sensor sensitivity, and wet well dimensions.
Lag pump on set point (LAGON)	The LAGON is used to turn the lag pump on at a designated level. This level is set on the basis of the following variables; inflow, downstream process flow acceptance, pump duty cycles, pump delivery rate, sensor sensitivity, emergency storage desired, and wet well dimensions.
High level alarm (HLA)	The HLA is used to detect high levels in the lift station and to notify maintenance staff via the SCADA system.
Current	High amperage can be a sign of motor wear, drag on the impeller and cutting blades, increased liquid levels in the lift station, resistance in the force main or valves, and restricted venting, etc. Low amperage can indicate impeller wear, low liquid levels in the lift station, or a leak in the force main.

The design procedure for lift station sizing and placement is summarized on Chart 5-2, and demonstrated in Ex. 4.

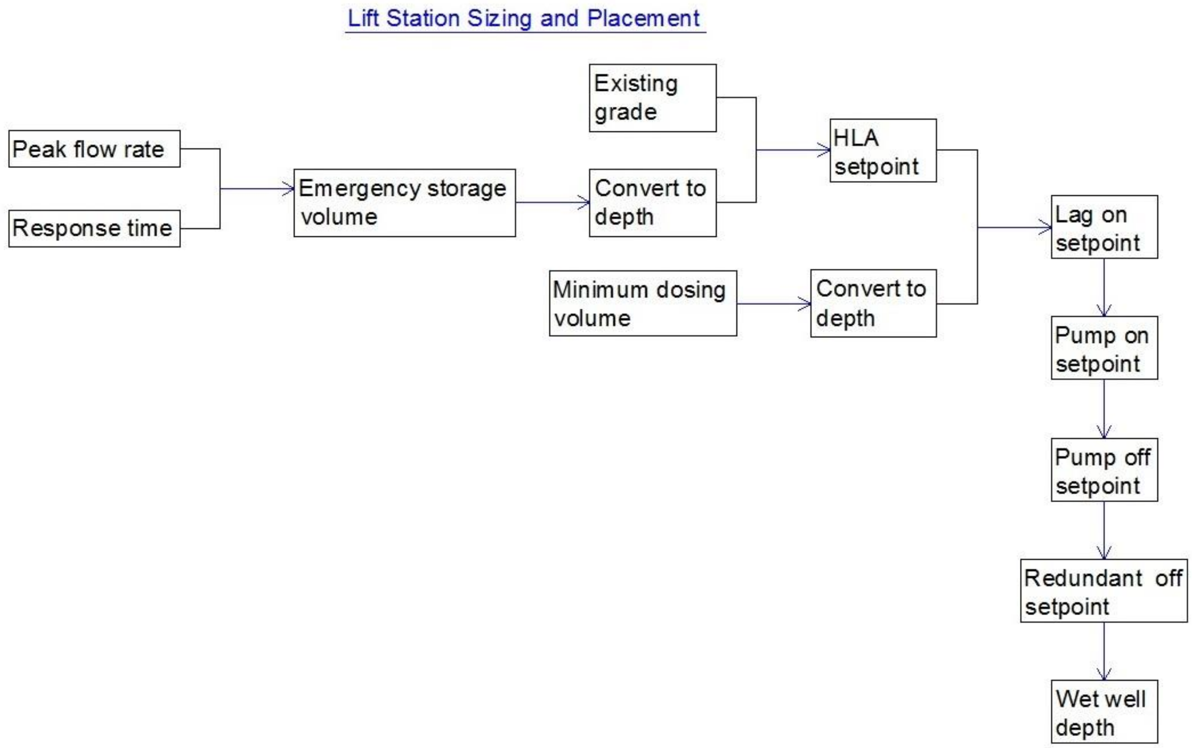

Chart 5-2
Lift station sizing and placement design flow chart

Example 4: Determine lift station sizing and control set point.

Using the flow data from Ex. 1, size a lift station to accommodate the SRRA described in the previous examples. The existing grade at the proposed lift station location is at 100 ft and the invert of the drainage collection line is 2 ft below exiting grade. Calculate the set points for the redundant off (RO), lead pump off (POFF). lead pump on (PON), lag pump on (LAGON), and high level alarm (HLA)

Assume a pump has been selected that will deliver 35 gal/min to the treatment facility. The minimum liquid level over the pumps is 0.8 ft and the total pump height is 2 ft. Assume liquid cover over the pumps at all times for corrosion resistance and pump cooling. To accommodate duplex pumps and a slide rail assembly and hatch access opening, the wet well must be at least 6 ft in diameter.

Solution

1. Calculate the emergency storage capacity desired. The emergency storage is based on the maintenance personnel response time anticipated for the SRRA. For this example, we will use a detection and response time of 3 hours. Use the future peak daily flow as the daily flow.
   a. Emergency storage = (future peak use, gal/d) (response time, d) = (4798 gal/d) (0.125 d) = 600 gal
2. Convert emergency storage to depth. Ignore the sloped sides at the bottom of the wet well because the emergency storage will be in the portion of the wet well with vertical sides.
   a. Compute the incremental volume
      Volume = π r^2h = π $(3\text{ ft})^2$ (1 ft) / (1ft) = 28.3 ft^3/ft
   b. Convert to gal/ft
      Volume = (28.3 ft^3/ft) (7.48 gal/ft^3) = 211.5 gal/ft
   c. Compute the depth required for emergency storage. Depth for emergency storage = (600 gal) / (211.5 gal/ft) = 2.8 ft
3. Calculate volume between pump start and stop levels (V_h) and convert to depth (d_h).
   a. For small lift stations, as used at SRRAs, the maximum pump start frequency is assumed to be 2 cycle per hour at peak hourly flow.
   b. The expected peak hourly flow, which is averaged over a 10 h period is: peak hourly flow = (4798 gal/d) / (10 h/d) = 480 gal/h
   c. For a maximum of 2 cycle per hour at peak flow, the estimated working volume between pump start and stop is:
      V_h = (480 gal/d) / (2 cycle/h) = 240 gal/cycle
   c. Compute depth (d_h) that corresponds to working volume V_h between pump start and stop.
      Depth for working volume (d_h) = 240 gal / 211.5 gal/ft = 1.1 ft
4. Calculate high level alarm (HLA) set point:
   HLA = existing grade – depth to invert of inlet – depth of emergency storage
   HLA = 100 ft – 2.0 ft – 2.8 ft = 95.2 ft

5. Calculate lag on (LAGON) set point:

   LAGON = HLA – (d_h)

   LAGON = 95.2 ft – (1.1 ft) = 94.0 ft

6. Calculate pump on (PON) set point:

   PON = LAGON – (d_h)

   PON = 94.1 ft – (1.1 ft) = 92.9 ft

7. Calculate pump off (POFF) set point:

   POFF = PON – (d_h)

   POFF = 93.0 ft – (1.1 ft) = 91.8 ft

8. Calculate redundant off (RO) set point:

   RO = POFF – pump height + minimum liquid level

   RO = 91.9 ft – 2.0 ft + 0.8 ft = 90.6 ft

9. Compute the wet well depth

   Wet well depth required = existing grade – RO + minimum liquid level

   Wet well depth = 100 ft – 90.7 ft + 0.8 ft = 10.2 ft

Comment

Value engineering will be necessary for this design to select a pumping system that provides the highest level of function at the least cost. There are a number of safety factors in this design, including (1) the use of future peak summer flow as the design parameter, and (2) a long response time with SCADA notification. This design is consistent with the common 10-ft deep wet well. Alternately, additional safety factors could be added by designing to a 12-ft nominal lift station depth for more remote facilities. Also note that there is additional emergency storage capacity above the invert of the inlet that is available but not accounted for.

5.4 OPERATION, MAINTENANCE, AND TROUBLESHOOTING

There are several operation and maintenance activities related to wastewater collection and pumping, including maintenance of fixtures, drain piping, control of trash and solids present in wastewater, and operation of the pumping system. A summary of maintenance activities is presented in Table 5-3.

Table 5-3
Operation, maintenance, and troubleshooting of wastewater collection and pumping systems at SRRA comfort stations

Maintenance item	Frequency	Time, h	Tools/materials	Estimated annual cost, $[a]
Clean drain lines	As needed	1	Drain cleaning tools. Chemical drain cleaners are not recommended	100
Respond to lift station alarms	As needed	1	Electrical test equipment, flashlight, tank access tools	300
Check lift station operation	Monthly	0.15	Flashlight	180
Remove solids from lift station	Annually	2	Service contract with pumping company	500
Replace pump	As-needed, expected pump life is 20 to 30 y	1	Replacement pump, plumbing tools and hardware	2500
Clean urinals and toilets	Daily	1	Cleaning equipment	Staff
Service pumps	Per vendor specifications	2	Plumbing tools and hardware	200

a. Estimated costs include labor and materials. Labor costs estimated assuming a rate of $100/hr.

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6.0 PRIMARY TREATMENT

Following wastewater collection, the first stage of wastewater treatment (i.e., primary treatment) is the removal of bulk solids and preliminary reduction of dissolved organic matter. Several technologies can be used to achieve the goals of primary treatment, including the septic tank, Imhoff tank, and anaerobic baffled reactor (ABR). At the Dunnigan SRRA pilot facility, primary treatment was accomplished using a custom designed ABR with integrated upflow sludge contactors and anaerobic filters. The layout, design procedure, and other considerations are presented below.

This section describes the approach for developing a primary treatment system design, as shown in Chart 6-1.

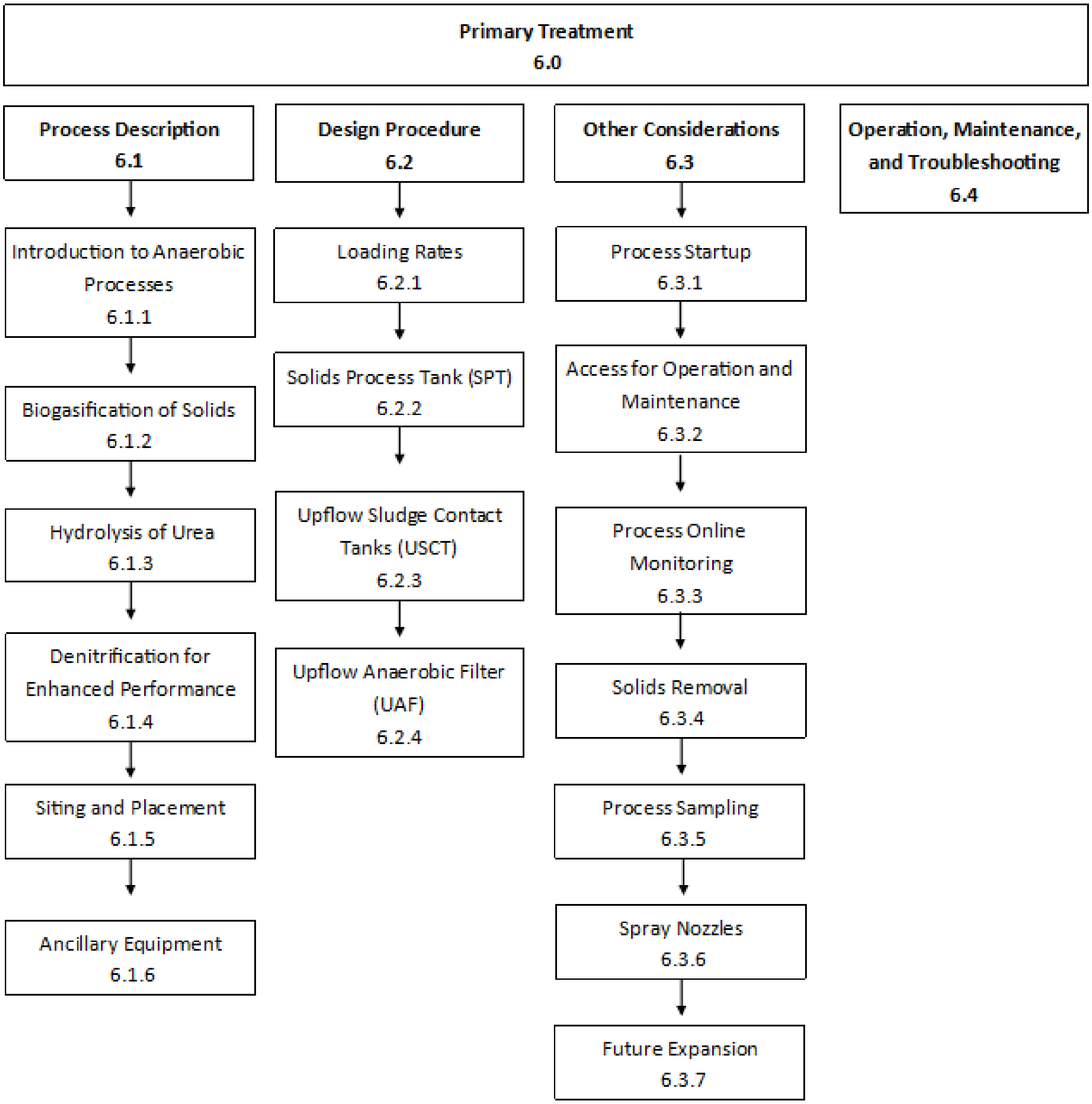
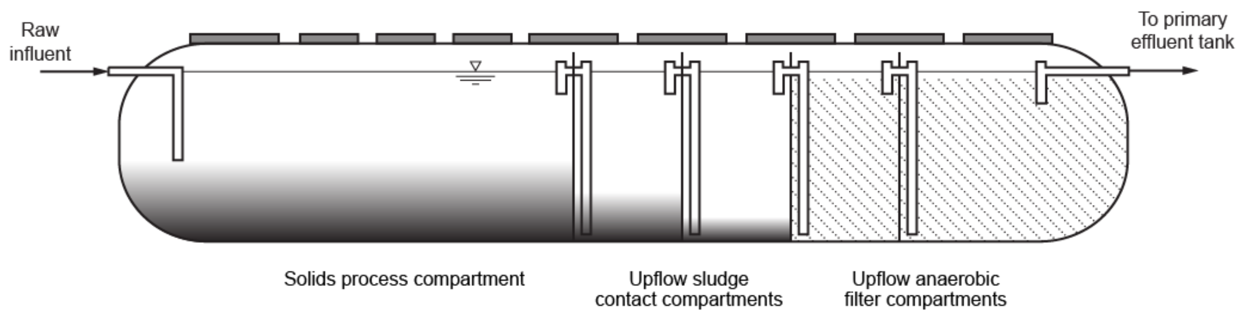


Chart 6-1
Primary treatment system design

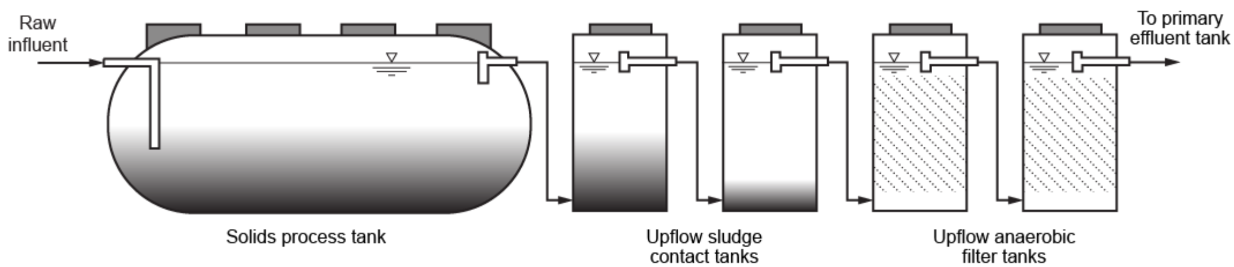
6.1 PROCESS DESCRIPTION

Because the process uses a multi-compartment design, it is possible to put all compartments into a single tank or to break the compartments up within interconnected stand-alone tanks, as shown on Fig. 6-1. The range of flow rates expected at SRRAs is

1500-8000 gal/d (see Fig. 3-4). Based on this range, primary tanks, typically constructed of fiberglass, are generally designed with a volume capacity of 10000-30000 gal.



(a)



(b)

Figure 6-1
Implementation of the primary treatment system in two alternative process configurations: (a) single-tank and (b) multiple tanks

The single-tank design requires less space and reduces costs compared with the multiple-tank design; however, operation of the multiple tank design is expected to be somewhat improved. The round cross-section used in the single-tank design is effective for the solids process tank, but is not ideal for the upflow compartments. Views of the alternate tank configurations are shown on Fig. 6-1.

6.1.1 Introduction to Anaerobic Processes

The anaerobic oxidation process can be described as a two-stage process: the first stage is identified as waste conversion (acetogenesis, acidogenesis), in which complex organics are first hydrolyzed and then fermented into simple organic compounds and volatile fatty acids (VFA) such as acetate by facultative bacteria known as acetogens and acidogens (McCarty, 1964; Speece, 1996; Bitton, 2005). After the organic matter has been converted to simpler compounds, waste stabilization (methanogens) takes place, where the acids are synthesized by methanogens into methane and carbon dioxide (McCarty, 1964). A key factor in the anaerobic oxidation process is the balance between the microorganisms responsible for each step. When the system is in equilibrium, the methanogens transform the acids at the same rate that acids are formed (McCarty, 1964). Therefore, high acid concentrations are an indication that the acid-forming bacteria and the methanogens are not in balance. As discussed later in this section, the development of scum in SRRA anaerobic tanks is an indication of unbalanced acidic conditions.

6.1.2 Biogasification of Solids

The two major mechanisms of methane formation are the breakdown of acetic acid, which is the most prevalent volatile acid produced in the fermentation of carbohydrates, proteins, and fats, and the reduction of carbon dioxide (McCarty, 1964; Bitton, 2005). The chemical reactions of methane formation are as follow:

1. Utilization of acetic acid:

$$CH_3COOH \rightarrow CH_4 + CO_2 \quad (6\text{-}1)$$

2. Reduction of carbon dioxide:

$$CO_2 + 8H \rightarrow CH_4 + 2H_2O \quad (6\text{-}2)$$

Growth and acid utilization rates of methane formers are slow, and are usually limiting factors in anaerobic treatment (Speece, 1996; Duncan and Horan, 2003). The methane-forming microorganisms are entirely anaerobic and even small amounts of oxygen can be very toxic. Methanogens are also sensitive to any environmental change, including temperature, organic loading, waste composition, and other factors (McCarty, 1964).

Solids that enter the tank, including human fecal waste and tissue paper, initially sink to the bottom of the tank where they contact an environment of active anaerobic fermentation. As the fresh waste begins to ferment, gas bubbles from metabolic activity form on the particles, causing the particles to become buoyant and rise through the liquid column. Solids that are not swept out of the tank during this variable buoyancy period eventually join with other floating solids to form a scum layer. Under normal conditions, it is expected that solids will tend to digest in the tank (e.g., are converted biologically to methane and carbon dioxide); however, the low pH resulting from the fermentation process (i.e., acidogenesis) inhibits further stages of anaerobic digestion.

Because of the slow growth rate of methanogens, conventional SRRA septic tanks are not effective for anaerobic degradation of wastewater solid constituents. In a conventional SRRA septic tank, solids accumulate in the scum layer, where methanogens are inhibited because of low pH, and dissolved constituents pass through the liquid zone without contacting anaerobic biomass that are primarily on the bottom of the tank. Thus, to enhance the performance of anaerobic systems, modifications to the conventional design are required. The modifications that were evaluated at the Dunnigan SRRA are (1) a spray system to break up the scum layer, (2) augmentation of influent wastewater with nitrate, (3) multi-compartment design with upflow hydraulics and sludge contact zone, and (4) integrated anaerobic filter compartments.

6.1.3 Hydrolysis of Urea

The wastewater derived from SRRA facilities contains a significant fraction of urine. Because urine has high urea [$NH_2(CO)NH_2$] content and urea decomposes through hydrolysis into ammonium and bicarbonate, it is important to understand the chemistry related to the presence of urea. The following reaction can be used to represent the action of microbial unease enzyme on urea.

$$NH_2(CO)NH_2 + H_2O + \text{urease} \rightarrow (NH_4)_2CO_3 \rightarrow \underset{14}{NH_4^+} + \underset{61}{HCO_3^-} \qquad (6\text{-}3)$$

As shown on Eq. (6-3), the production of bicarbonate relative to ammonium from urea hydrolysis is 4.36 g HCO_3^- / g NH_4^+-N (61 / 14). The alkalinity production in terms of calcium carbonate is shown on Eq. (6-4).

$$\text{Alkalinity} = \left(\frac{4.36\,\text{g HCO}_3^-}{\text{g NH}_4^+\text{-N}}\right)\left(\frac{50\,\text{g CaCO}_3}{\text{equivalent}}\right)\left(\frac{\text{equivalent}}{61\text{g HCO}_3^-}\right) = \frac{3.57\,\text{g CaCO}_3}{\text{g NH}_4^+\text{-N}} \qquad (6\text{-}4)$$

The conversion of urea to ammonium and bicarbonate is known to occur within a few hours, depending on temperature. In the study conducted at the Dunnigan SRRA, about half of the urea was converted in the wastewater pumped over from the lift station, while the remainder of the urea underwent hydrolysis in the first compartment of the ABR.

6.1.4 Denitrification for Enhanced Performance

Because of the problems with a pure anaerobic treatment approach, a process configuration was developed to utilize denitrification reactions to improve the performance of the primary treatment system. Denitrification occurs under anoxic conditions (i.e., without free dissolved oxygen present), and requires nitrate or nitrite (the alternate electron acceptor) and a supply of biodegradable organic carbon or alternate electron donor. The bacteria that convert nitrate or nitrite to nitrogen gas are known as facultative aerobes, which are able to complete respiration reactions using oxidized nitrogen only when dissolved oxygen is limited (Rittmann and McCarty, 2001). The heterotrophic bacteria (e.g., Pseudomonas, Bacillus) use organic compounds, such as those present as BOD in wastewater. Conceptually, the implementation of denitrification-type treatment occurs when nitrate present in the recycled water is returned to the primary tank after it is used for fixture flushing. Benefits of denitrification in the anoxic tank include (1) degradation of particulate and soluble organic constituents in the primary tank, (2) alkalinity production to buffer downstream nitrification reactions, and (3) reduced methane emissions to the atmosphere. The relevant chemical reactions, derived from Rittmann and McCarty (2001), are as follows:

$$\underset{14}{\text{NO}_3^-} + 0.12\,\underset{24.2}{\text{C}_{10}\text{H}_{19}\text{O}_3\text{N}} \rightarrow$$

$$0.5\text{N}_2 + 0.05\text{C}_5\text{H}_7\text{O}_2\text{N} + 0.29\text{H}_2\text{O} + 0.88\text{CO}_2 + \qquad (6\text{-}5)$$

$$0.07\text{NH}_4^+ + \underset{17}{1\text{OH}^-} + \underset{4.2}{0.07\text{HCO}_3^-}$$

In Eq. (6-5), $C_{10}H_{19}O_3N$ is used to represent organic constituents present in wastewater. If it is assumed that the BOD demand of wastewater is systems with long solids retention time (SRT) is about 2 g O_2 / g $C_{10}H_{19}O_3N$, the amount of BOD consumed in the denitrification reactions is 3.5 g BOD / g nitrate-N (2 x 24.2 / 14). As shown on Eq. (6-5), hydroxide is produced as a result of the denitrification reaction at a rate of 1.21 g OH^-/g NO_3-N (17 / 14). The reaction shown on Eq. (6-6) describes the production of bicarbonate alkalinity from the hydroxide byproduct.

$$H_2CO_3 + 5CO_2 + \underset{102}{6OH^-} \rightarrow H_2O + \underset{366}{6HCO_3^-} \qquad (6\text{-}6)$$

As shown in Eq. (6-6), 1 mole of bicarbonate is produced for each mole of hydroxide ion, or 3.59 g HCO_3^-/g OH^- (366 / 102), which equates to 4.36 g HCO_3^-/ g NO_3-N reduced (1.21 x 3.59). In addition, 0.3 g HCO_3^-/ g NO_3-N reduced (4.2 / 14) is produced directly in the denitrification reaction. The overall impact on alkalinity due to denitrification in systems with long SRT is estimated to be 4.66 g HCO_3^-/ g NO_3-N reduced. The overall production of alkalinity, expressed in terms of calcium carbonate equivalents, is shown on Eq. (6-7).

$$\text{Alkalinity} = \left(\frac{4.66\text{ g } HCO_3^-}{\text{g } NO_3^-\text{-N}}\right)\left(\frac{50\text{ g } CaCO_3}{\text{equivalent}}\right)\left(\frac{\text{equivalent}}{61\text{ g } HCO_3^-}\right) = \frac{3.8\text{ g } CaCO_3}{\text{g } NO_3^-\text{-N}} \qquad (6\text{-}7)$$

It is important to note that there is a significant demand for BOD for the denitrification reaction and in some cases the BOD demand will exceed the measured BOD that is available. It is expected that a portion of the oxygen demand that is not measured using the BOD test will be available as a carbon source in the denitrification reaction because (1) the BOD test is not representative of conditions in the primary tank, and (2) organic compounds that are not readily biodegradable initially will hydrolyze and become bioavailable over time in the primary tank. Therefore, the BOD value may underestimate the total amount of carbon available for denitrification.

6.1.5 Siting and Placement

Under ideal circumstances, the primary treatment system will be placed in an area where monitoring and maintenance can be performed satisfactorily. In general, the minimum practical cover should be used after considering (if applicable) traffic loads and potential for freezing. Monitoring is facilitated by the tank being located as close to the surface as possible such that the water level is readily accessible. Flow moves through the primary tanks by hydraulic pressure, therefore, little or no slope is required to facilitate water movement.

Shallow tanks facilitate monitoring activities such as sludge and scum measurements, sample collection, and installation of sensors and other equipment for remote monitoring. Tank maintenance, including removal of accumulated solids, is also accomplished more effectively when the tank contents are closer to the ground surface. In reviewing the placement of existing septic tanks, some tanks were found at depths exceeding 10 ft. At these depths, solids removal and tank monitoring were challenging. There should be sufficient space around all process tanks so that they can be accessed for tank cleaning and maintenance. In addition to space, a hose bib that can provide water at 20 gal/min and 50 lb/in^2 should be provided to help maintain the tanks. Traffic loading and setback distances should follow manufacturer and building code specifications.

6.1.6 Ancillary Equipment

Some type of ventilation system should be provided to control the accumulation of hazardous and odorous gases. When primary tanks are connected directly to the building plumbing, it is possible to use the building vent stack to disperse these gases into the atmosphere. In the case where a lift station is used to deliver flow, independent systems will be needed to manage gases. A system that uses a standpipe can be used to vent gases in areas where odors are not a concern. If odors are a potential issue, the standpipe system can be used to vent gases through an activated carbon filter. Many carbon filter systems are commercially available or a suitable system can be constructed onsite.

6.2 DESIGN PROCEDURE

The purpose of primary treatment is to remove settleable solids and a portion of the colloidal and dissolved organic material from raw wastewater. Primary effluent that is low in solids and organic matter reduces the loading on downstream processes, making them more effective for nitrogen removal and production of recycled water. Reducing the load on downstream processes also reduces the operation and maintenance requirements in some cases.

6.2.1 Loading Rates

The hydraulic and solids loading rates are both important to overall performance. The hydraulic loading rate is important because it has an impact on the removal of fine settleable solids. The solids loading rate is important because most of the suspended solids that enter the tank are stored until the tank is cleaned during service visits. While some fraction of influent solids is digested in the tank, the high solids loading results in an elevated rate of solids accumulation compared with a lightly loaded tank. As described previously, at the high loading rate, the solids are subject to acid fermentation, which results in conditions that inhibit methanogenic digestion.

6.2.2 Solids Process Tank (SPT)

The operation of the SPT is similar to that of a standard septic tank, except that a few modifications are made to improve operation and maintenance. The modifications include using spray nozzles to help break up the scum layer, using sensors to detect the amount of solids in the tank and the temperature, and recycling high nitrate loads to help digest the solids that accumulate in the tank.

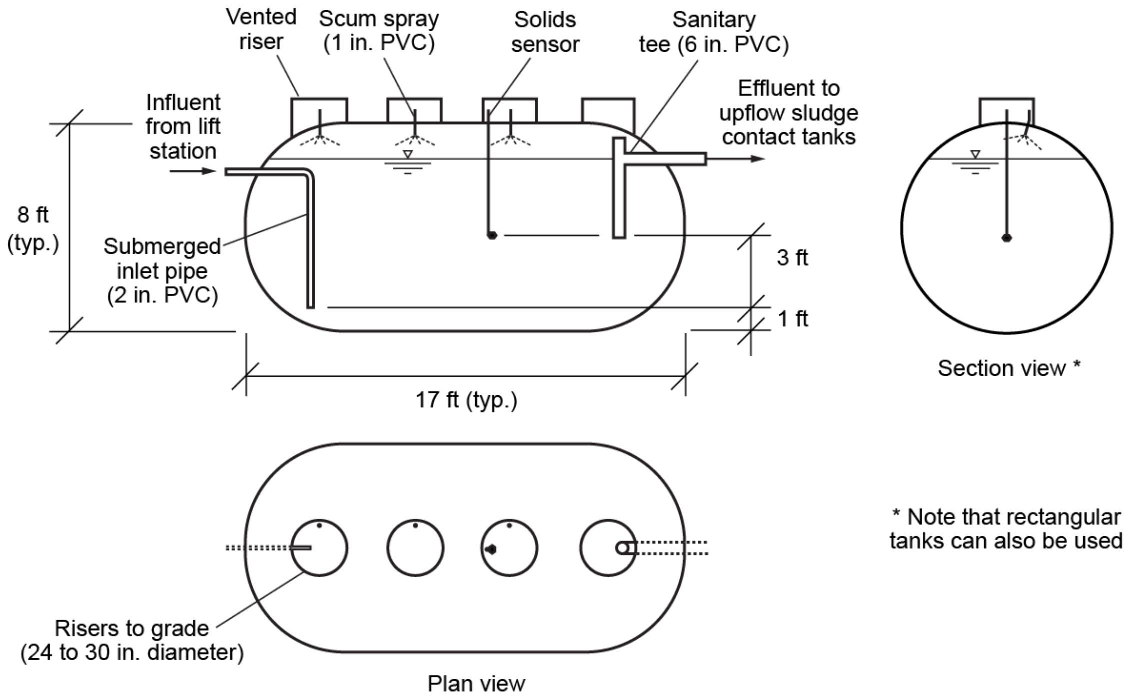


Figure 6-2
Views of solids process tank

Types of COD are removed using two mechanisms: (1) COD associated with settled solids, and (2) COD resulting from anoxic denitrification. As shown in Table 6-1, particulate matter in the influent accounts for about 40 percent of the COD loading.

Table 6-1
Summary of constituent ratios in raw SRRA wastewater from grinder pump station and at each stage of anaerobic treatment at the Dunnigan SRRA

	Ratio for indicated process			
Parameter	Raw	SPT	USCT	UAF
TSS/COD	0.39	0.24	0.20	0.14
BOD/COD	0.42	0.48	0.43	0.30
fCOD/COD	0.38	0.50	0.56	0.60

On average, about 27 percent of the influent COD is removed when the SPT is designed with an effective hydraulic retention time (HRT) of one day. The denitrification of nitrate present in the recycled water results in additional COD removal, beyond the COD removed from simple settling. In testing at the Dunnigan SRRA pilot facility, the COD removal increased to 41 percent when nitrate was added to the influent water. The denitrification reaction is expected to also consume some of the COD associated with the solids that accumulate in the tank. For purposes of modeling performance when recycled water is used for flushing, the values used in Table 6-2 are recommended.

Table 6-2
Summary of expected performance of the SPT with potable and recycled water with high nitrate concentration used for flushing

		Mean removal in SPT with 24 h HRT at 20 degrees Celsius (C), %	
Parameter	Abbreviation	Potable water supply	Nitrified recycled supply
BOD	$SPT_{BOD,S}$	21	48
TSS	$SPT_{TSS,S}$	51	62
COD	$SPT_{COD,S}$	27	42
sCOD	$SPT_{sCOD,S}$	-	47

The removal rate for COD and BOD in the anaerobic treatment train needs to be corrected for temperature and HRT. Based on the relationship shown on Fig. 6-3 from data collected at the Dunnigan SRRA, and Eq. (6-10), the removal rate can be adjusted for different water temperatures and HRT.

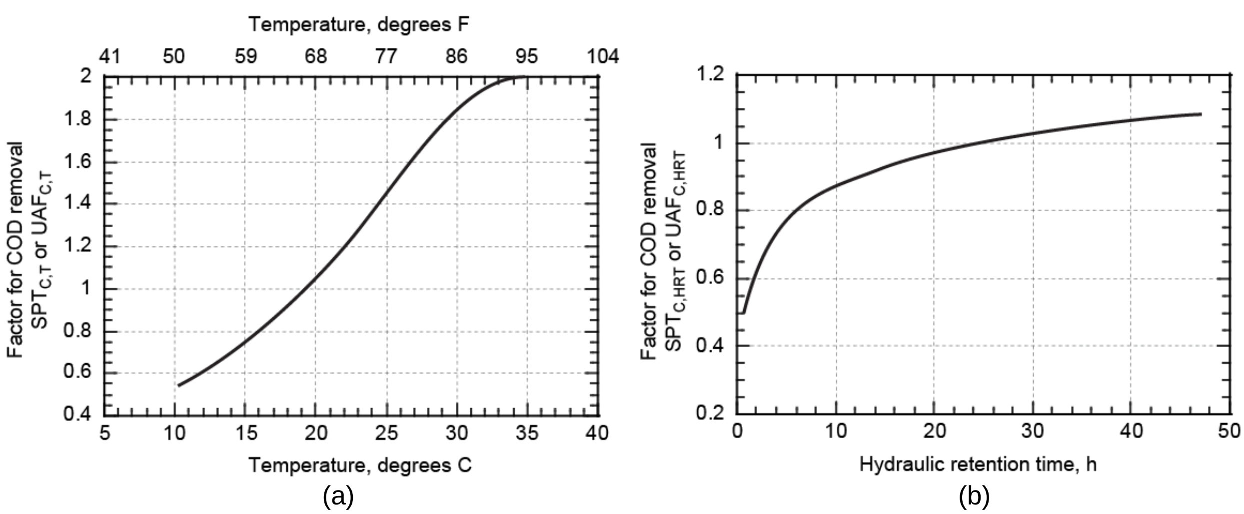


Figure 6-3
Generalized correction factors for COD removal in anaerobic systems:
(a) temperature, and (b) HRT

The TSS removed in the SPT accumulates as settled (sludge) and floating (scum) solids in the tank. Because of the action of acid fermentation of the solids, nearly all solids will occur as a floating scum mat under conditions where recycled water is not used for flushing. High-nitrate water has not been used for fixture flushing and the impacts on the SPT are not known. When potable water is used for flushing, solids accumulate at the rate shown on Eq. (6-8):

$$S_A = 3.6 \text{ gal} / 1000 \text{ pe} \tag{6-8}$$

where S_A = solids accumulation rate, gal/1000 pe

The primary role of the SPT is to remove and store solids. Sizing of the PST is based on providing adequate retention time for solids settling with some additional volume for solids storage. The following equations can be used for sizing and predicting the performance of the SPT:

$$SPT_V = (HRT_{mean})\,(Q_{mean,fut}) + (S_A)(PE_{fut}/1000)(S_R) \tag{6-9}$$

where

SPT_V = SPT volume required, gal
HRT_{mean} = mean hydraulic retention time desired, d (typically 1 d)
$Q_{mean,\,fut}$ = mean flowrate based on future flow projections, gal/d
PE_{fut} = expected number of people based on future design, capita/d
S_R = Time between solids removal events, d

The performance of the SPT can be determined using the following equation:

$$SPT_{COD,R} = (SPT_{COD,S})\,(SPT_{C,T})\,(SPT_{C,HRT}) \tag{6-10}$$

where

$SPT_{COD,R}$ = expected removal of COD in SPT at HRT and temperature, percent
$SPT_{COD,S}$ = standardized COD removal in SPT (see Table 6-2), percent
$SPT_{C,T}$ = correction factor for COD removal in SPT as a function of temperature (see Fig. 6-3a)
$SPT_{C,HRT}$ = correction factor for COD removal in SPT as a function of HRT (see Fig. 6-3b)

The application of Eqs (6-8) through (6-10) is shown on Ex. 5.

6.2.3 Upflow Sludge Contact Tanks (USCT)

The USCTs shown on Fig. 6-4 are used following the SPT. Their primary function is to remove additional TSS and BOD prior to the UAFs. The basic principle of operation is that sludge particles that carry over from the SPT are discharged into the bottom of the first USCT. The upward velocity of flow in the USCT is lower than the settling velocity of the sludge particles. Therefore, the sludge particles accumulate at the bottom of the tank and form a sludge bed, also referred to as a sludge blanket.

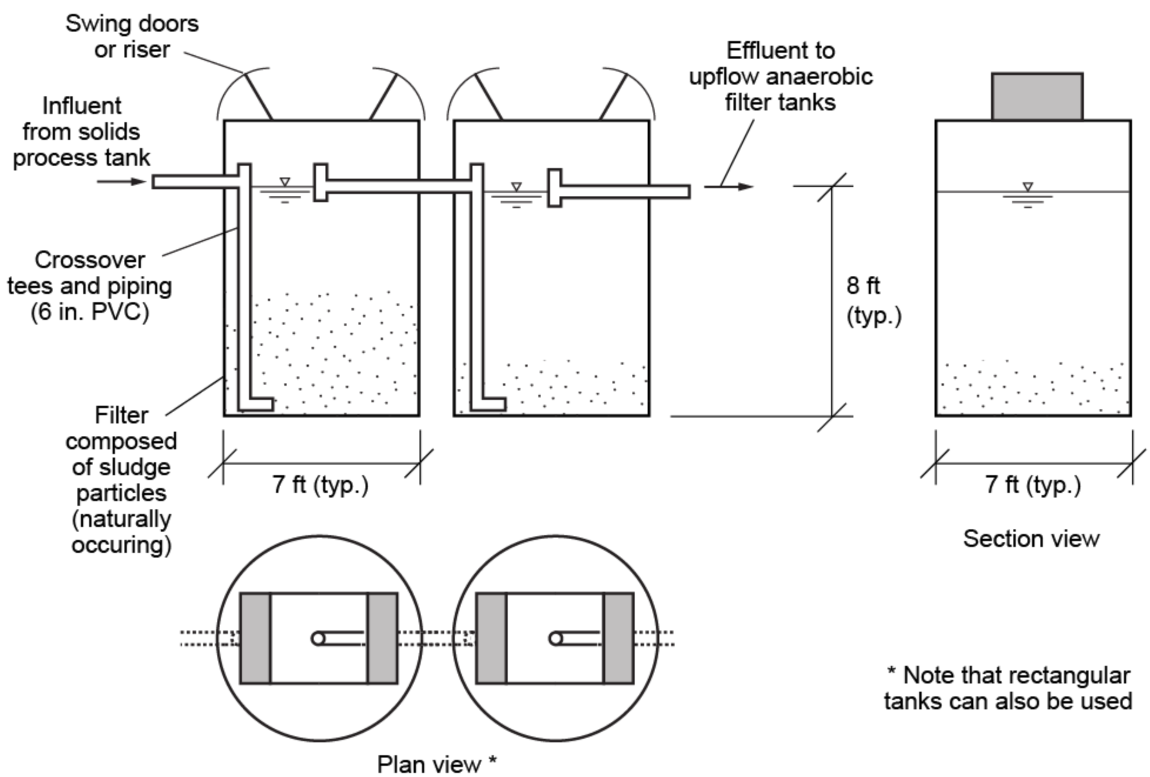


Figure 6-4
Views of upflow solids contact tanks

The standard design consists of two tanks in series with an upflow velocity of 3 ft/h (1 m/h) under peak flow conditions (Sasse, 1998). Peak flow conditions are computed using the design peak daily flow spread over a period of 10 h. The 10-h value represents the time period over which almost all water use occurs, based on observations at numerous SRRA facilities. A USCT depth of 6 ft provides adequate storage capacity for long-term operation. The Dunnigan SRRA pilot facility USCT operated for three years without needing any solids pumping and without significant sludge accumulation.

$$USCT_{area} = (Q_{peak,fut}) / [(7.48)\ (T_{DP})\ (V_u)] \qquad (6\text{-}11)$$

where

$USCT_{area}$ = surface area of each USCT, ft^2
$Q_{peak,fut}$ = design future peak flow, gal/d
7.48 = conversion factor, ft^3/gal
T_{DP} = time period of the diurnal peak, h/d
V_u = maximum upflow velocity, ft/h

Using the design specified above, the results obtained at the Dunnigan SRRA are summarized in Table 6-3. Note that the removals reported in Table 6-3 are primarily the result of additional TSS reduction and were not found to be sensitive to temperature or other seasonal variations.

Table 6-3
Summary of results for USCT at the Dunnigan SRRA with one-day HRT, two tanks in series, and operation at 20 degrees C

Parameter	Mean removal, %
BOD	10
TSS	15
COD	6
sCOD	7

6.2.4 Upflow Anaerobic Filter (UAF)

The final stage of the anaerobic treatment system is the UAF. The UAF is a tank that is filled with an inert packing material, as shown on Fig. 6-5. The packing material is typically made of plastic, with a high surface area for the growth of microbial communities and a high void fraction so that clogging cannot occur. Similar to the operation of the USCT, flow enters the bottom of the UAF reactor, flows upward through the packing, and exits at the top of the reactor.

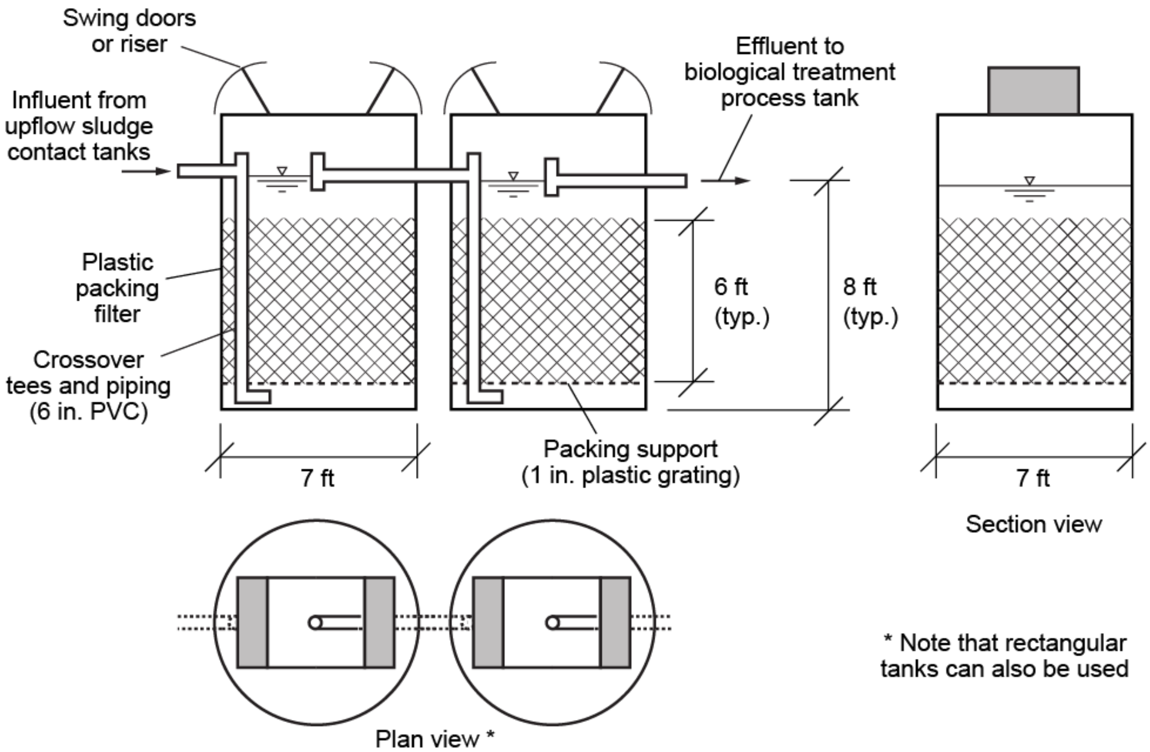


Figure 6-5
Views of upflow anaerobic filters

As with the USCT, the maximum upflow velocity through the packing should not exceed 3 ft/h. Modern plastic packing has a void fraction exceeding 90 percent and specific surface areas between 30 and 60 square feet per cubic foot (ft^2/ft^3) (100 and 200 square meters per cubic meter [m^2/m^3]). Images of alternative packing materials are shown on Fig. 6-6. It is expected that after some period of service, the packing material may need to be removed for cleaning or replacement. At the Dunnigan SRRA pilot facility, media were placed into mesh bags to facilitate removal. In later experiments, pieces of packing were tied together with string. Both of these techniques were effective in improving the handling of the packing. Note that mesh bags should have openings that are on the same order of magnitude as the void openings in the packing so that flow is not restricted by blinding of the mesh material. Because higher surface area packing can support more anaerobic biomass, a correction factor is applied to adjust for COD removal for different packings. The packing should rest on a grating so that a plenum is created for influent distribution. The plenum height should be 6 to 12 inches.

(a) (b) (c)

Figure 6-6
Examples of packings used in anaerobic filters:
(a) Bio-Pac with a specific surface area of 30 ft^2/ft^3 and void fraction of 95 percent,
(b) Bio-Rings with a specific surface area of 32 ft^2/ft^3 and void fraction of 95 percent,
and (c) Bio-Pac media placed randomly in UAF tank

Note: Bio-Pac media was utilized at the Dunnigan SRRA pilot study site, but a reference to the product is not a recommendation or endorsement of the product. Specifications of comparable media may vary.

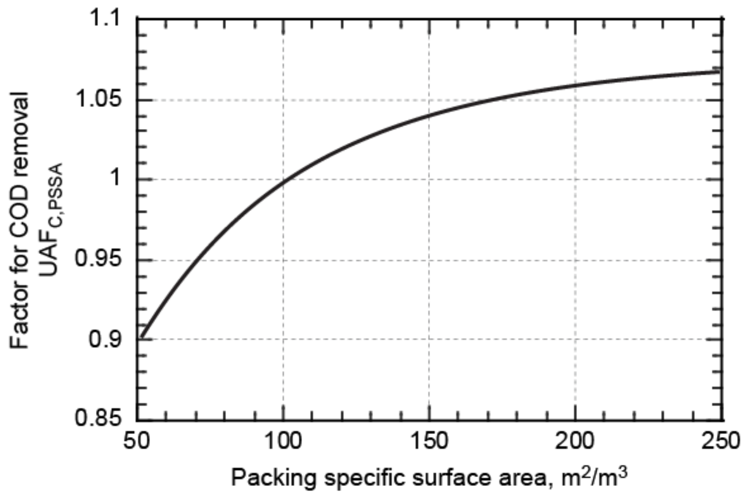


Figure 6-7
Correction factor for COD removal in anaerobic filter
for packing specific surface area (after Sasse, 1998)

The surface area of the UAF can be computed using Eq. (6-12).

$$UAF_{area} = (Q_{peak,fut}) / [(7.48)\ (T_{DP})\ (V_u)\ (P_V)] \tag{6-12}$$

where

$USCT_{area}$ = surface area of USCT, ft^2
$Q_{peak,fut}$ = design future peak flow, gal/d
7.48 = conversion factor, ft^3/gal
T_{DP} = time period of the diurnal peak, h/d
V_u = max upflow velocity, ft/h
P_V = void fraction of the packing

Table 6-4
Summary of results for UAF at the Dunnigan SRRA with one-day HRT, two tanks in series, Bio-Pac packing, and operation at 20 degrees C

Parameter	Mean removal, %
BOD	10
TSS	28
COD	24
sCOD	27

The performance of the UAF can be determined using Eq. (6-13).

$$UAF_{COD,R} = (UAF_{COD,S})\ (UAF_{C,T})\ (UAF_{C,PSSA})\ (UAF_{C,HRT}) \tag{6-13}$$

where

$UAF_{COD,R}$ = expected removal of COD in UAF for 1-d HRT and given temperature and packing, percent
$UAF_{COD,S}$ = standardized COD removal in UAF (see Table 6-4), percent
$UAF_{C,T}$ = correction factor for COD removal in UAF as a function of temperature (see Fig. 6-3a)
$UAF_{C,PSSA}$ = correction factor for COD removal in UAF as a function of packing specific surface area (see Fig. 6-7)
$UAF_{C,HRT}$ = correction factor for COD removal in UAF as a function of HRT (Fig. 6-3b)

The design procedure for primary treatment system design and the application of Eqs. (6-12) and (6-13) are summarized on Chart 6-2, and demonstrated in Ex. 5.

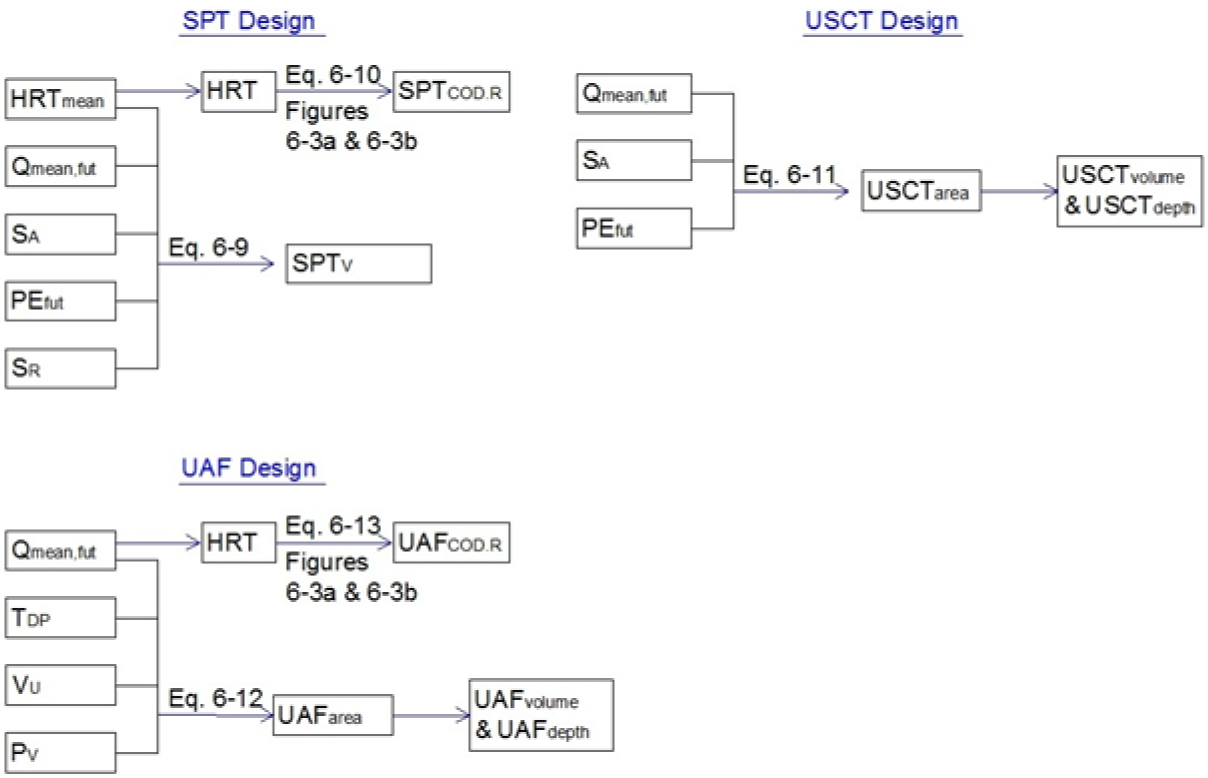


Chart 6-2
Aerobic treatment system design flow chart

Example 5: Design procedure for anaerobic treatment system.

The SRRA facility located in the Central Valley region and described in Ex. 1 is being considered for water reuse. Using the data findings presented in Ex. 1 and the data derived in Exs. 2 and 3, develop the anaerobic process design. In the design, consider diurnal fluctuations and seasonal impacts on usage and temperature. The toilets will be flushed with well-nitrified recycled water. Solids will be removed from the SPT on an annual basis. Assume the bio-pac media is to be used in the UAF.

Solution

1. Determine the size and performance of the SPT using Eqs. (6-9) and (6-10)
   a. Tank size:
      $SPT_V = (HRT_{mean})\ (Q_{mean,fut}) + (S_A)(PE_{fut})(S_R)$
      $SPT_V = (1\ d)\ (2399\ gal/d) + (3.6\ gal/1000\ pe)(1859\ pe)(365\ d) = 4842\ gal$
   b. SPT performance (winter):
      $SPT_{COD,R} = (SPT_{COD,S})\ (SPT_{C,T})\ (SPT_{C,HRT})$
      $HRT_{winter} = (2399\ gal) / [(2068\ gal/d)] = 1.16\ d = 28\ h$
      $SPT_{COD,R} = (42\%)\ (0.51)\ (1.03) = 22\ \%$
   c. SPT performance (summer):
      $HRT_{summer} = (2399\ gal) / [(2783\ gal/d) = 0.88\ d = 21\ h$
      $SPT_{COD,R} = (42\%)\ (1.82)\ (0.98) = 75\ \%$
2. Determine the size and performance of the USCT using Eq. (6-11)
   a. Tank size:
      $USCT_{area} = (Q_{peak,fut}) / [(7.48)\ (T_{DP})\ (V_u)]$
      $USCT_{area} = (4798\ gal/d) / [(7.48)\ (10\ h/d)\ (3\ ft/h)] = 21\ ft^2$
      $USCT_{volume} = (\text{liquid depth + freeboard})\ (USCT_{area}) =$
      $(6\ ft + 2\ ft)\ (21.4\ ft^2) = 171.2\ ft^3 = 1279\ gal$
   b. USCT performance:
      From table 6-3, $USCT_{COD,R} = 6\ \%$
3. Determine the size and performance of the UAF using Eqs. (6-12) and (6-13)
   a. Tank size:
      $UAF_{area} = (Q_{peak,fut}) / [(7.48)\ (T_{DP})\ (V_u)\ (P_V)]$
      $UAF_{area} = (4798\ gal/d) / [(7.48)\ (10)\ (3\ ft/h)\ (0.95)] = 23\ ft^2$
      $UAF_{volume} = (\text{liquid depth + freeboard})\ (UAF_{area})$
      $= (6\ ft + 2\ ft)\ (22.5\ ft^2) = 180\ ft^3 = 1347\ gal$
      Note, liquid volume = (1347 gal) (95 %) = 1279 gal
   b. UAF performance, winter:
      $UAF_{COD,R} = (UAF_{COD,S})\ (UAF_{C,T})\ (UAF_{C,PSSA})\ (UAF_{C,HRT})$
      $HRT_{winter} = (2)\ (1310\ gal) / (2068) = 1.27\ d = 30\ h$
      $UAF_{COD,R} = (24\%)\ (0.51)\ (1.04)\ (1.) = 13\ \%$

c. UAF performance, summer:

$UAF_{COD,R} = (UAF_{COD,S})\ (UAF_{C,T})\ (UAF_{C,PSSA})\ (UAF_{C,HRT})$

HRT_{summer} = (2) (1279 gal) / (2783) = 0.92 d = 22 h

$UAF_{COD,R}$ = (24%) (1.82) (0.99) (1) = 43 %

4. Compute effluent constituent concentrations for organics and suspended solids using the findings from the preceding steps and the relationships given in Table 6-1.

		Process effluent, mg/L					
		SPT		USCT		UAF	
Constituent	Influent	Winter	Summer	Winter	Summer	Winter	Summer
COD	1384	1080	352	1015	331	887	189
TSS	504	259	85	203	66	124	26
BOD	524	518	169	436	142	266	57
sCOD	498	540	176	568	185	532	113

Comment

As demonstrated in this example, the process temperature has a significant impact on process performance.

6.3 OTHER CONSIDERATIONS

In addition to the items discussed above that are related to process sizing and performance, a number of practical issues need to be addressed that are related to process startup, operation, maintenance, and future expansion.

6.3.1 Process Startup

Anaerobic biomass can require months to develop under warm conditions, and much longer if process startup happens under cold conditions. Because a number of microbial communities are expected to be active in the primary tank, it is recommended that seed biomass (inoculum) be used. During the initial process startup, biomass from an anaerobic digester can be added at a rate of about 5 percent by volume. At startup of the Dunnigan SRRA pilot facility ABR, about 200 gal of biomass from an anaerobic digester in operation at the University of California, Davis were added. If nitrate recycle to the primary tank is expected, biomass from the anoxic zone of a municipal wastewater treatment facility can be used as a process seed.

6.3.2 Access for Operation and Maintenance

Various maintenance activities are associated with the primary treatment system, including observations of sludge and scum accumulation, adjustment of the spray system (if used), collection of water samples for analysis, installation and calibration of sensors, removal of accumulated solids, and installation and removal of packing (UAF only). To accomplish these activities effectively, it is recommended that an all season access drive be provided for service vehicles. In addition, a hose bib rated for 20 gal/min at 50 lb/in^2 and connected to the potable or nonpotable water supply is needed for service operations. The SPT, USCT, and UAF processes are not placed under a canopy and do not require any protection from weather, because they are subsurface tanks designed for exposure to various climatic conditions.

6.3.3 Process Online Monitoring

The primary operation for process monitoring is the measurement of sludge and scum, temperature, and flowrate in the SPT and USCTs. Solids monitoring is accomplished using an ultrasonic or sonar type device that is submerged in the tank. A signal is directed up for scum detection and down for sludge. Floating scum accounts for nearly all solids in the SPT, while nearly all solids in the USCTs occur as sludge. The solids monitoring device should be compatible with the SCADA system to facilitate remote monitoring. In the testing conducted at the Dunnigan SRRA pilot facility, readings from online sludge and scum monitoring systems were in agreement with physical measurements made using a core-type sampling device (see Fig. 6-8). Note that sludge from primary tanks is typically non-granulated. The granulated sludge shown on Fig. 6-8b is unusual.

(a) (b)

Figure 6-8
Manual monitoring of sludge accumulation in ABR: (a) sludge sampling device, and (b) sample of sludge collected from bottom of first compartment

6.3.4 Solids Removal

Periodically, solids should be removed from the tank before they have a significant impact on process hydraulics. A standard pumping truck with adequate holding capacity can be used for scum removal, as shown on Fig. 6-9. In general, when the solids in the SPT reach about 30 percent of the tank volume, it is recommended to remove only the floating solids to the extent possible. The rationale behind the removal of scum only is that the scum is largely composed of acidic byproducts of fermentation that inhibit anaerobic decomposition, while the solids on the bottom consist of active biomass that should be retained in the tank. There should be no need to remove solids from the USCT or UAF unless the solids reach about 50 percent of the volume, which never occurred in three years of operation at the Dunnigan SRRA pilot facility. If solids removal from the USCT or UAF is necessary, about 6 to 12 inches should be left in the tank as inoculum.

Figure 6-9
Truck used to remove floating solids from ABR

6.3.5 Process Sampling

Liquid flows from one compartment to the next through cross-over tees. Cross-over tees are used so that individual process compartments can be drained independently without draining the all process compartments simultaneously. The cross-over tees also provide a suitable sample location and allow individual process tanks to be isolated. Periodic sampling of water quality samples is recommended to benchmark performance in the summer and winter seasons for organic, nutrient, and solid constituents.

6.3.6 Spray Nozzles

The Dunnigan SRRA pilot facility included an anaerobic recycle through spiral-jet type spray nozzles placed in the SPT compartment of the ABR. The spray system was used to control the formation of a scum layer at the access ports and near the sludge and scum monitoring equipment. The normal dosing system for the ABR spray system was an on-time of 1 min at a frequency of 24 cycles per day. It was also found that operation of the spray system for longer periods of time increased the velocity of flow through the ABR and resulted in degraded performance. At the Dunnigan SRRA pilot facility, four spiral-jet nozzles (Model 3/4 HHSJX-PVC 120 210; see Table 7-3 for detailed specifications) were mounted about 12 inches above the liquid level surface and were able to maintain about a 1-inch-diameter zone with limited scum accumulation (see Fig. 6-10), even when the mean scum thickness was at 30 inches. The clear zone facilitated better accuracy in measuring the sludge and scum thickness, and created an area for gases to escape, reducing the buoyancy of the scum layer.

Figure 6-10
Area kept relatively free of floating solids using spray nozzle
(Clear area improves monitoring capabilities.)

6.3.7 Future Expansion

There should be no need to expand the process unless the mean flowrate exceeds the design values significantly. At the Dunnigan SRRA pilot facility, the hydraulic retention time was found to have a clear impact on performance. When the fixtures were switched from high flow to low flow, the retention time in the ABR effectively doubled. The increased HRT from about 3 to almost 7 d reduced the effluent constituent concentrations of BOD and TSS by about 50 percent. If process expansion is required to enhance treatment or to meet increased water usage, additional USCT or UAF processes can be added in line. If process expansion is anticipated, space can be reserved for these tanks.

6.4 OPERATION, MAINTENANCE, AND TROUBLESHOOTING

The primary treatment system will require limited regular maintenance to ensure long-term stable operation. The activities associated with operation, maintenance, and troubleshooting are summarized in Table 6-5.

Table 6-5
Operation, maintenance, and troubleshooting summary for primary treatment tanks at SRRA facilities.

Maintenance item	Frequency	Time, h	Tools/materials	Estimated annual cost, $[a]
Manual solids measurement	6 month (mo)	2	Sludge sampling device, scum probe	400
Tank observation	1 mo	1	Hand pump, probe, handheld meter	1200
Corrosion check	1 mo	0.25	Flashlight to check for corrosion on any metallic objects	300
Tank venting and lid security check	6 mo	0.25	None	50
Solids removal	12 – 24 mo	4	Service contract	2500

a. Estimated costs include labor and materials. Labor costs estimated assuming a rate of $100/hr.

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7.0 AEROBIC TREATMENT

The effluent from the anaerobic filters is processed using aerobic filters to remove ammonium and residual organic compounds, and to prepare the water for sand filtration. Four aerobic treatment systems were evaluated for the Dunnigan SRRA pilot study: Advantex packed-bed filter (PBF), plastic media trickling filter, Anua membrane bioreactor (MBR) with submerged aeration, and backwashing filter (BWF) units. Any of the aerobic treatment systems could be used to meet the treatment goals described in this manual, and each process is subject to certain advantages and disadvantages. However, the BWF was selected for further testing because it has a small footprint, low energy input, and reliable performance.

This section describes the approach for developing an aerobic treatment system design, as shown in Chart 7-1.

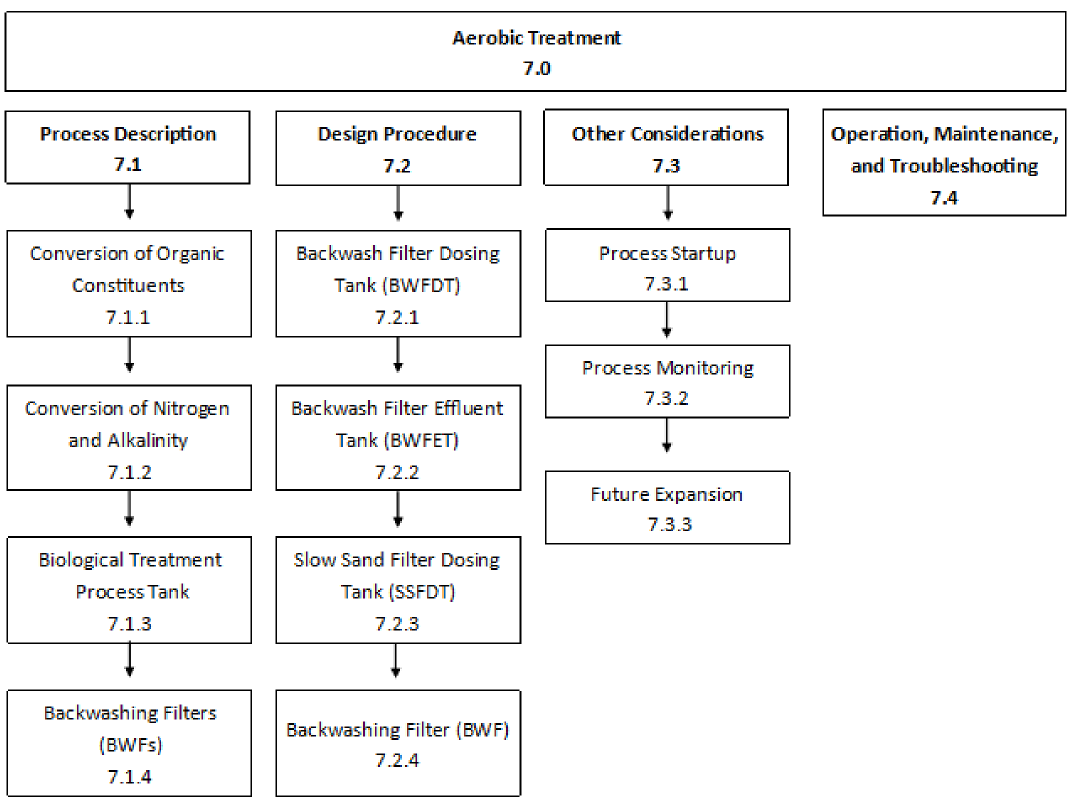


Chart 7-1
Aerobic treatment system design

The basic process for the aerobic treatment system is shown on Fig. 7-1. The primary system components are the biological treatment process tank (BTPT) and the BWF process tanks.

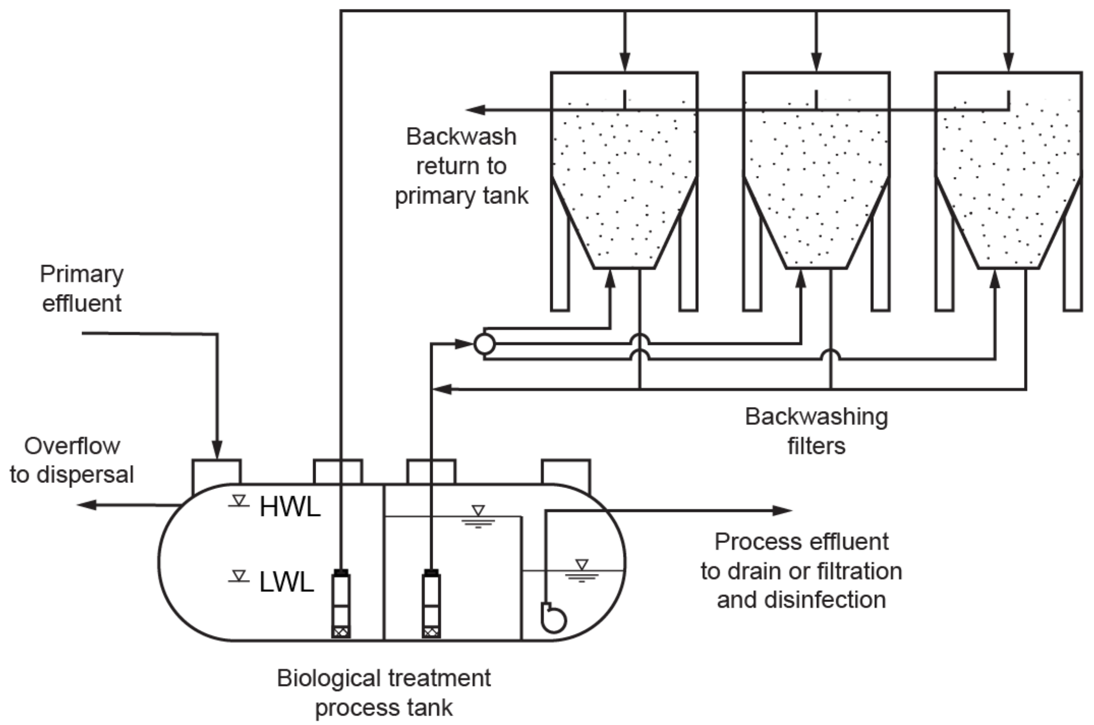


Figure 7-1
Components of aerobic treatment process (schematic diagram)

7.1 PROCESS DESCRIPTION

Several process configurations are possible, depending on the site constraints and availability of custom tanks. Aerobic treatment includes:

- Conversion of organic constituents.
- Conversion of nitrogen and alkalinity.
- Biological treatment process tank (BTPT) for storage and equalization of effluent from the anaerobic process and backwash filter. This water is also used for dosing filter backwash.
- Backwashing filters (BWFs) is to remove residual COD, ammonium nitrogen, and particulate matter from the anaerobic effluent.

Discussed in this section are process descriptions, siting and layout considerations, and ancillary systems.

7.1.1 Conversion of Organic Constituents

The aerobic conversion of organic compounds (i.e., COHNS) in wastewater, as shown on Eq. (7-1), is driven by a mixed microbial community within the wastewater treatment process. The general reactions shown on Eqs. (7-1) and (7-2) are for the degradation of organic matter (i.e., BOD and COD) and the degradation of cell biomass (i.e., $C_5H_7NO_2$), respectively. The reaction given in Eq. (7-2) is known as endogenous respiration and represents the consumption of biomass within the process, which is an important element of filters such as the BWF. Such endogenous respiration reactions are used so that the process does not generate a waste sludge stream that is common with activated sludge and MBR type processes.

$$COHNS + O_2 + \text{nutrients} \rightarrow CO_2 + NH_3 + C_5H_7NO_2 + \text{other products} \qquad (7\text{-}1)$$

$$C_5H_7NO_2 + O_2 \rightarrow 5CO_2 + 2H_2O + NH_3 + \text{energy} \qquad (7\text{-}2)$$

7.1.2 Conversion of Nitrogen and Alkalinity

In addition to the removal from and/or conversion of organic constituents in the wastewater, the BWF is also used to convert ammonium to nitrate. This conversion is known as nitrification, which is among the most important functions of the BWF because the nitrate is used subsequently to enhance the performance of the primary treatment system (i.e., SPT, USCT, and UAF). As shown on Eq. (7-3), the nitrification reaction requires aerobic conditions and produces nitrate and hydrogen ions. The hydrogen ion reacts with and consumes carbonate alkalinity. If the alkalinity is depleted fully, the pH will drop below about 6.5, which starts to inhibit further nitrification. Nitrification stops completely at pH 4.

$$NH_4^+ + 1.92O_2 + 0.08CO_2 \rightarrow$$
$$0.98NO_3^- + 0.016C_5H_7NO_2 + 0.95H_2O + 1.98H^+ \qquad (7\text{-}3)$$

$$6H^+ + 6HCO_3^- \rightarrow H_2CO_3 + 5CO_2 + 5H_2O \qquad (7\text{-}4)$$

Based on reactions shown on Eq. (7-3) and (7-4), 1 mole of bicarbonate is consumed for each mole of hydrogen ion neutralized, or 8.63 g of HCO_3^-/g N oxidized. The stoichiometric alkalinity requirement, expressed commonly as calcium carbonate ($CaCO_3$), is computed as follows.

$$\text{Alkalinity} = \left(\frac{8.63\,\text{g}\ HCO_3^-}{\text{g}\ NH_4^+\text{-N}}\right)\left(\frac{50\,\text{g}\ CaCO_3}{\text{equivalent}}\right)\left(\frac{\text{equivalent}}{61\text{g}\ HCO_3^-}\right) = \frac{7.07\,\text{g}\ CaCO_3}{\text{g}\ NH_4^+\text{-N}} \qquad (7\text{-}5)$$

Using the relationship shown on Eq. (7-5), the amount of alkalinity, as $CaCO_3$, required to buffer the pH change during nitrification is 300 mg/L of ammonia nitrogen.

$$\text{Alkalinity required} = (300\text{ mg N/L})(7.07\text{ mg }CaCO_3\text{/mg N}) = 2120\text{ mg/L} \qquad (7\text{-}6)$$

As described in Secs. 3 through 5, the primary sources of alkalinity in SRRA wastewater when operated in a water reuse configuration are urea hydrolysis and nitrate reduction.

7.1.3 Biological Treatment Process Tank (BTPT)

The BTPT has a number of functions that are essential to the operation of the process. The BTPT is divided into three compartments: the backwash filter dosing tank (BWFDT), the backwash filter effluent tank (BWFET), and the slow sand filter dosing tank (SSFDT). Each of these tanks has important functions. The BWFDT is used for storage and equalization of effluent from the anaerobic process and as a dosing tank for the BWF, and a timer-controlled pumping system used for dosing the BWF is located in the BWFDT. The BWFET is used for collection and storage of backwash filter effluent, and also has a timer-controlled pumping system. The backwash pumps pump BWF effluent to the BWF for backwashing the filters; water that is not used for filter backwash overflows into the SSFDT. The range of flow rates expected at SRRAs is 1500-8000 gal/d (see Fig. 3-4). Based on this range, BTPT tanks are generally designed with a volume capacity of 6000-10000 gal. The overflow to the SSFDT is demand-dosed to the SSFs using a separate pumping system.

7.1.4 Backwashing Filters (BWFs)

The purpose of the BWF is to remove residual COD, ammonium nitrogen, and particulate matter from the anaerobic effluent. The BWF is characterized as an intermittently dosed, unsaturated-flow, biological filter. While this type of filter is not necessarily unique in the industry, the backwash mechanism allows it to operate at loading rates that are an order of magnitude higher than those of comparable systems that are commercially available. The filters contain a granular pumice packing that has a high surface area for biofilm growth and is easy to backwash.

Spray nozzles are used to distribute the water to be treated over the top of the packing. A single pump with a timer control cycles on and off to deliver even, metered dosing over the filter surface. As the water travels through the pores in the packing, a bacterial film (biofilm) adsorbs constituents from the wastewater for energy and to support growth. Clean water exits at the bottom of the filter through an underdrain assembly. The high dosing frequency mode of operation results in rapid and robust biofilm growth. Before the biofilm growth fills in the pores in the filter packing, a backwash cycle is used to disturb the bed and scour a portion of the biofilm from the pumice packing. If operated without the backwash cycle, the filters would begin to clog after about 2 d. Effluent from the backwash cycle is returned to the anaerobic SPT for treatment.

7.2 DESIGN PROCEDURE

Design of the BTPT and BWF requires consideration of flow variability, both diurnal and seasonal, and operational requirements of the treatment system. The design procedure is described below and then presented as a design example.

7.2.1 BWFDT

Primary effluent is discharged to the BWFDT on an as-produced basis. The BWFDT has an overflow for excess flow. A duplex pump system is used to apply water to the BWF from the BWFDT (see Fig. 7-2).

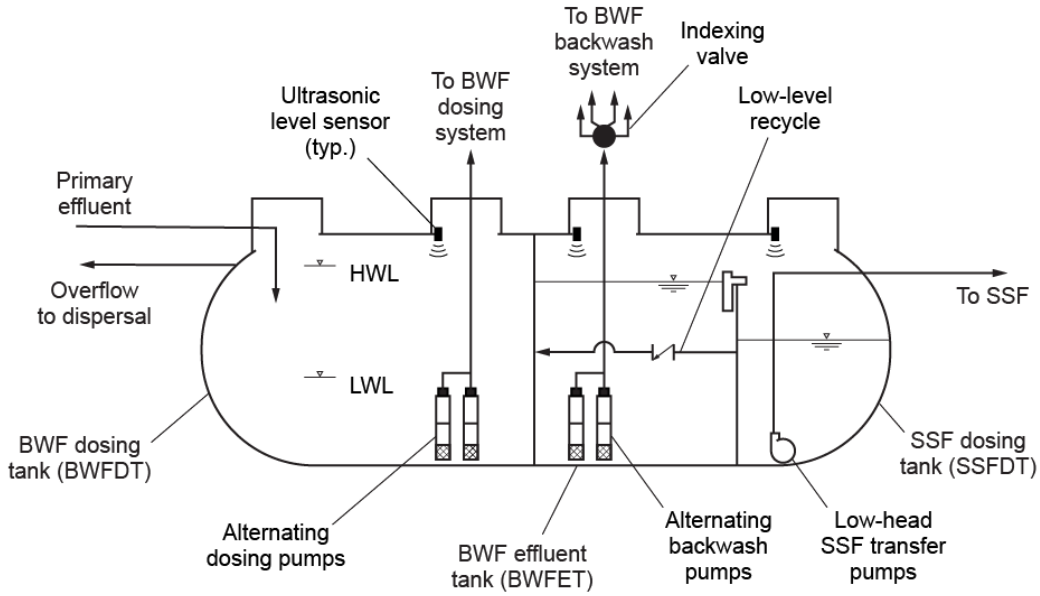


Figure 7-2
Components of biological treatment process tank (BTPT)

The BWFDT is also used to store primary effluent so that the flow can be metered out at a constant rate. Based on an analysis of long-term flow variability at the Dunnigan SRRA, the recommended working volume for the BWFDT is equal to one day of retention for the expected future summer-season flowrate (Qfut,summer), as shown on Eq. (7-7). Using this design basis, the BWFDT will be somewhat oversized for current usage, but will have capacity for increasing usage in the future. This tank size will also make it possible to provide treatment for the full flow for current conditions, and is suitable for 90 percent water recovery under future conditions where usage is increased by a factor of 30 percent.

$$BWFDT = (Q_{fut,summer})(1\ d) \tag{7-7}$$

The design of the BWF dosing pump depends on the number of filters to be dosed and is discussed below.

7.2.2 BWFET

The effluent from the BWF drains to the second compartment of the BTPT. This BWFET is used for intermediate storage of water that is available for backwashing the filters. A duplex pump system that is contained in the BWFET comes on as programmed for backwash operations. Each filter is backwashed about once daily and each event consists of a 4-min event with a flowrate around 60 gal/min. The filters are backwashed sequentially, so only one backwash pump system is required. The recommended working volume of the BWFET is 1000 gal, which provides extra capacity if additional backwash cycles are run.

7.2.3 SSFDT

The BWFET overflows to the SSFDT. The size of the SSFDT is based on the need to periodically decant the head from the SSF back to the BTPT during SSF maintenance activities. In general, about 500 gal of water would be returned from each SSF during cleaning. Assuming that no more than two filters would be serviced at one time, the SSFDT should have a working capacity of at least 1000 gal; however, a minimum working volume of 1500 gal is recommended. A low-head transfer pump is used to move water from the SSFDT to the SSF tanks. The control system for the SSF dosing pump is an on-demand, float-based system. It is also recommended that a piped drain with a check valve be installed from the SSFDT to the BWFDT. The purpose of this return line is to recycle a portion of the filter effluent during low-flow conditions, which are expected to occur during the cold season when treatment kinetics are also reduced.

7.2.4 BWF

The BWF is a commercial device developed by Dr. Rob Beggs. The filters are fabricated using mostly polyethylene because of its resistance to the abrasion caused by movement of the pumice within the filter. Loading on the BWF is based on the experience obtained at the Dunnigan SRRA pilot facility. Mass conversion rates for organic and nitrogenous constituents are summarized in Table 7-1 for specific loading conditions. Performance observations from the Dunnigan SRRA are presented on Fig. 7-3 and design performance values are summarized in Table 7-2.

Table 7-1
Summary of design parameters for BWF systems

Component or parameter	Unit	Value
Dosing frequency	1/d	720
Dose time	sec	5
Hydraulic loading	gal/d•filter	varies
Organic conversion	g COD/d	750
Nitrogen conversion (assumed)	g N/d	500 (summer) 300 (winter)
Packing	—	Grade 4 pumice (14 to 40 mesh)
Backwash frequency	1/d	1
Backwash time	min	4
Packing volume	gal	500
Packing bulk density	lb/ft^3	29
Packing specific gravity	—	1.13
Expected performance		
Ammonium N	mg N/L	< 1
COD	mg/L	< 50
BOD	mg/L	< 10
TSS	mg/L	< 10
Turbidity	NTU	< 2

Table 7-2
Summary of design constituent removal for BWF

	Percent Removed	
Constituent	Winter	Summer
BOD	98	99
COD	97	97
TSS	99	99
TDS	NR[a]	NR[a]
Nitrate-N	–[b]	–[b]
Ammonium-N	98	99
TN	NR	NR
TP	NR	NR
Alkalinity	–[c]	–[c]

a. Assume that there is no change in TN, TDS, or TP

b. Production of nitrate-N is proportional to ammonium-N removal

c. Alkalinity is removed as given in Eq. (7-5)

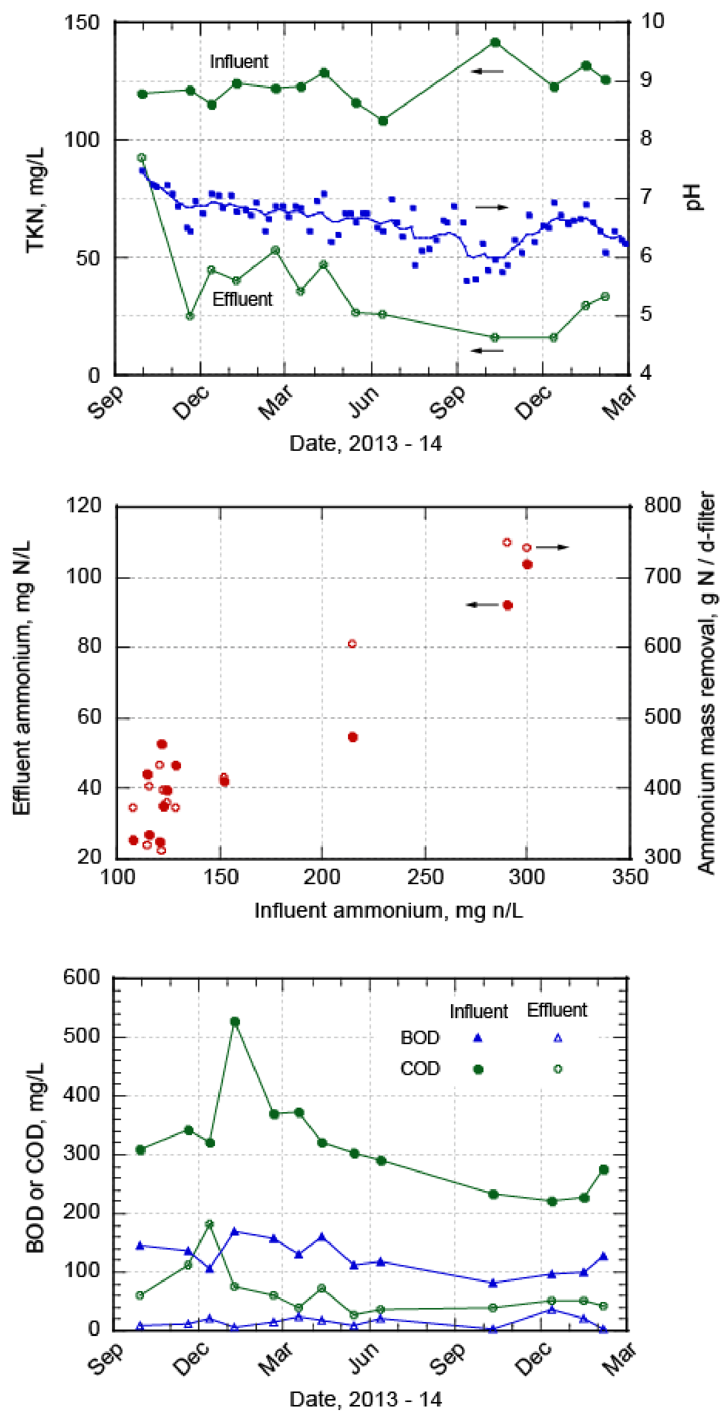


Figure 7-3
Performance data for BWF system at the Dunnigan SRRA

As shown on Fig. 7-4, several elements of the BWF need to be considered during the design process: the dosing system, the packing material, and the backwash system.

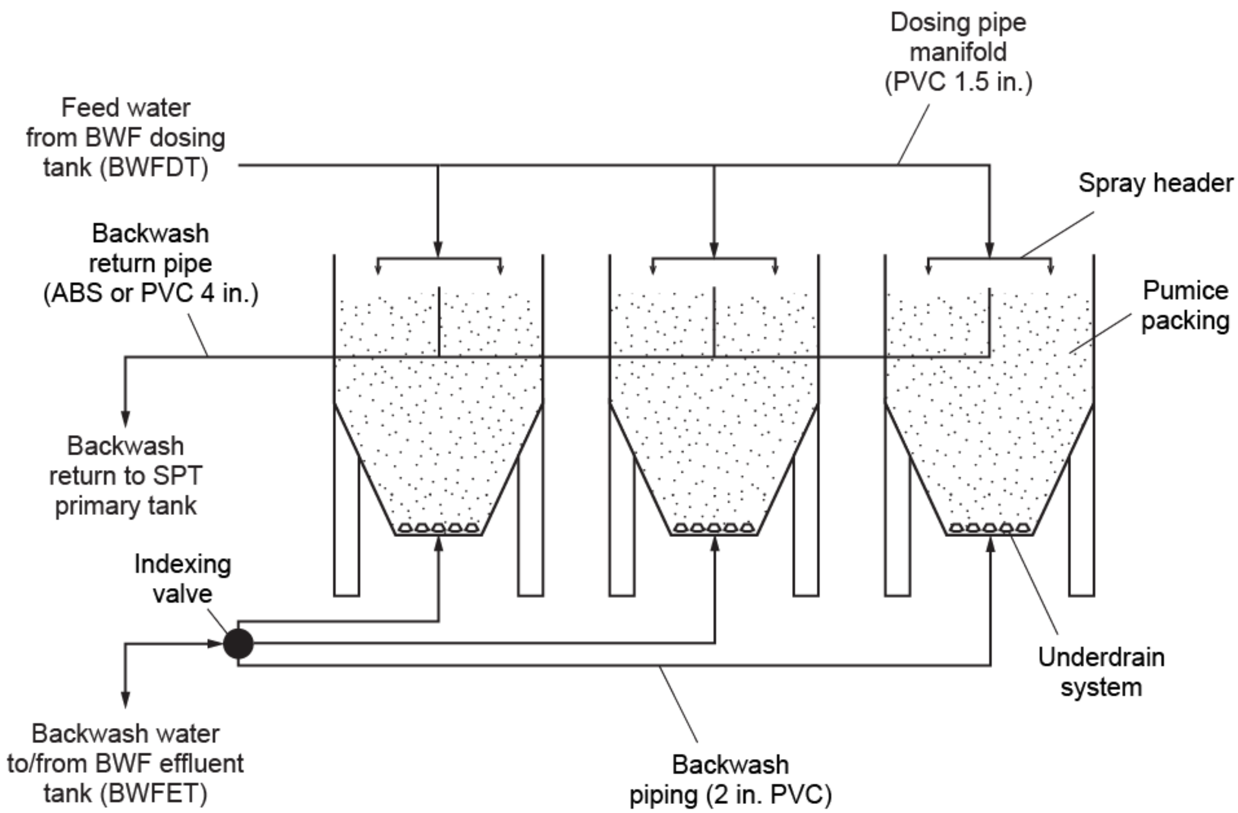


Figure 7-4
Profile view of BWF filter configuration

The dosing system consists of four spray nozzles oriented above the packing so that the spray pattern is relatively uniform across the surface. A single pump is used to simultaneously dose the spray nozzles in each BWF. The nozzles are 3/8-inch spiral jet, full-cone types, as shown on Fig. 7-5. The specific nozzle chosen will depend on the design of the filter and the dosing system. The nozzles used in the Dunnigan SRRA pilot project are 3/8 HHSJX–PVC 120 82, which requires 8.2 gal/min at 10 lb/in^2. A summary of alternative spray nozzles is given in Table 7-3. The Dunnigan SRRA pilot project used 2 filters for a total of 8 nozzles and a pump that could deliver about 64 gal/min. For filter systems that use configurations with more than two filters, a larger dosing pump, multiple dosing pumps, or spray nozzles with reduced capacity should be used. The BWF housing has a volume of 500 gal (plan area of 4 ft x 4 ft and height of 7 ft) with a cone angle of 60 degrees.

(a)

(b)

(c)

Figure 7-5
Images of dosing system installed in BWF:
(a) HHSJX PVC spiral jet nozzle, (b) dosing system, and (c) BWF installation

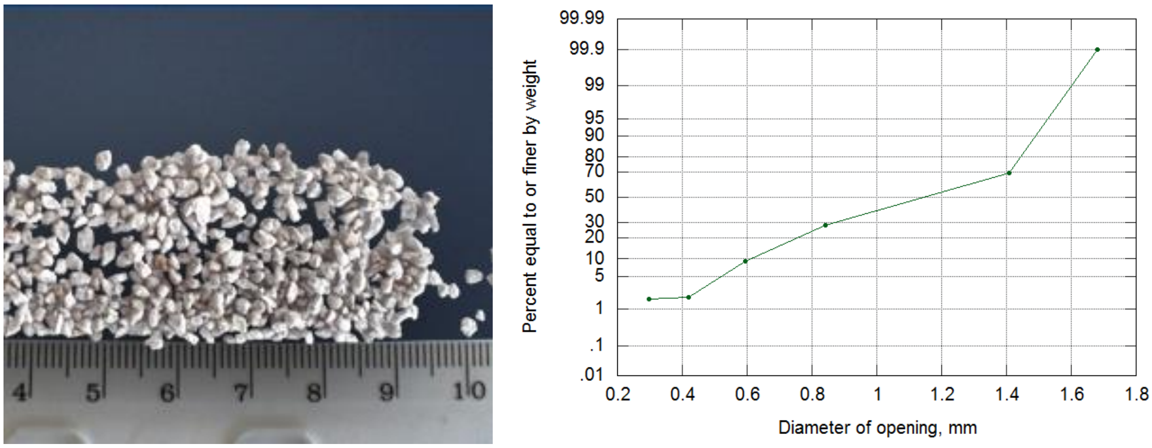
The packing material is mined and graded pumice. Pumice is lightweight and has a low particle density, making it easy to backwash; in addition, pumice has a high surface area and can support a large microbial community. The Dunnigan SRRA pilot facility filters used Grade 4 and Grade 6. It was found that the filter with Grade 4 packing performed better than the filter with Grade 6 packing. Images of the packing material and material specifications are shown on Fig. 7-6. A small fan is used on each filter to ventilate waste gases and supply oxygen to the bed; the fan is oriented to pull air into the bed from the surface and exhaust it through the underdrain.

Table 7-3
Specifications for alternative HHSJX spray nozzles
with spray angles of 90 and 120 degrees

		Capacity, gal/min at given pressure, lb/in^2			
Thread size, inch	Orifice size, inch	10	20	40	100
0.375	0.188	3.0	4.2	6.0	9.5
	0.219	4.0	5.7	8.0	12.6
	0.250	5.3	7.5	10.6	16.8
	0.313	8.2	11.6	16.4	26
0.50	0.375	12.0	17.0	24	38
	0.438	16.4	23	33	52
0.75	0.500	21	30	42	66
1.0	0.625	34	48	68	108
	0.750	47	66	94	149
1.5	0.875	64	91	128	202
	1.000	82	116	164	259
	1.125	96	136	192	304
2.0	1.375	140	198	280	443
	1.500	178	252	356	563

Source: Spraying Systems Co., www.spray.com

The internal backwash system is used to fluidize and mix the packing, which abrades and removes a portion of the biofilm. Biofilm and particulate matter that are dislodged from the packing are decanted from the filter with the backwash overflow water. The filters are backwashed once per day in sequence, using an indexing valve. With the indexing valve, a single-pump system can be used to backwash all filter units in the group. If more than six filters are in the group, multiple indexing valves can be used with independent pump systems. Excess backwash water drains back to the BWFET, so the pump systems should not contain any check valves or other features that would inhibit the return flow. The pump requirements for the backwash filter are 60 gal/min at 25 lb/in^2.



(a) (b)

Figure 7-6
BWF packing: (a) view of pumice, and (b) sieve analysis of grade 4 pumice

The design procedure for an aerobic treatment system design is summarized on Chart 7-2, and demonstrated in Ex. 6.

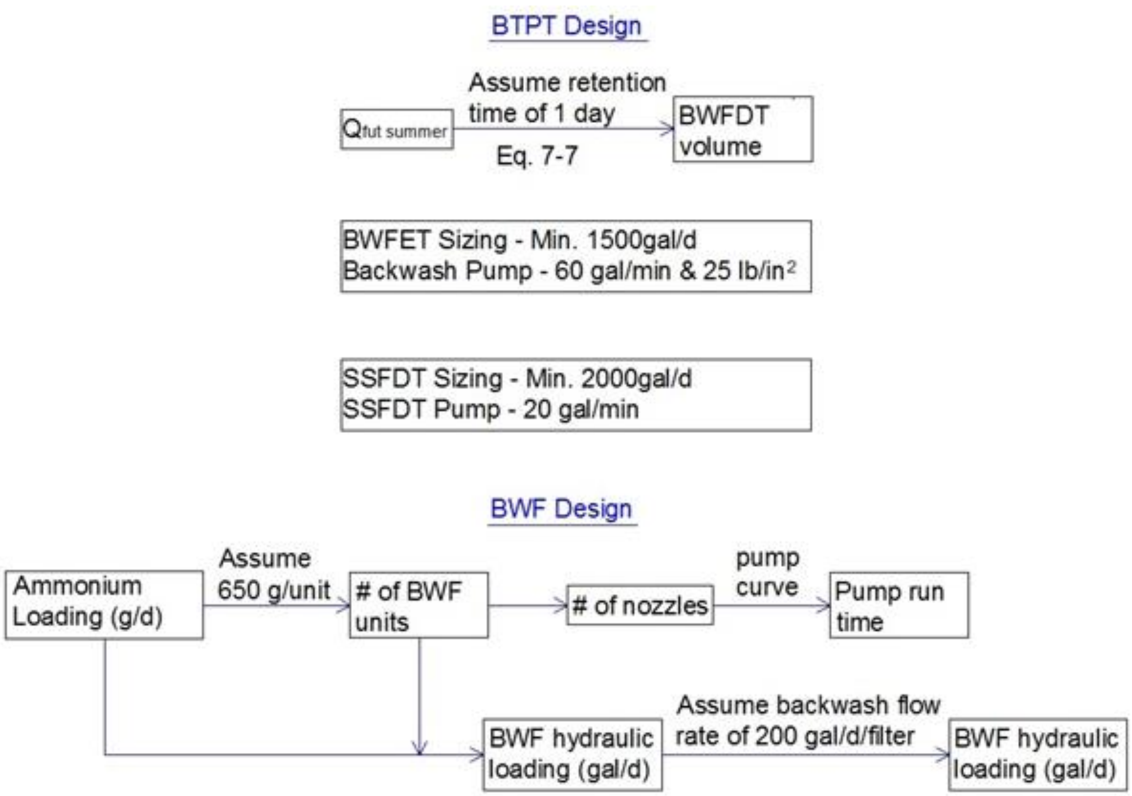


Chart 7-2
Aerobic treatment system design flow chart

Example 6: Design the BTPT and BWF system.

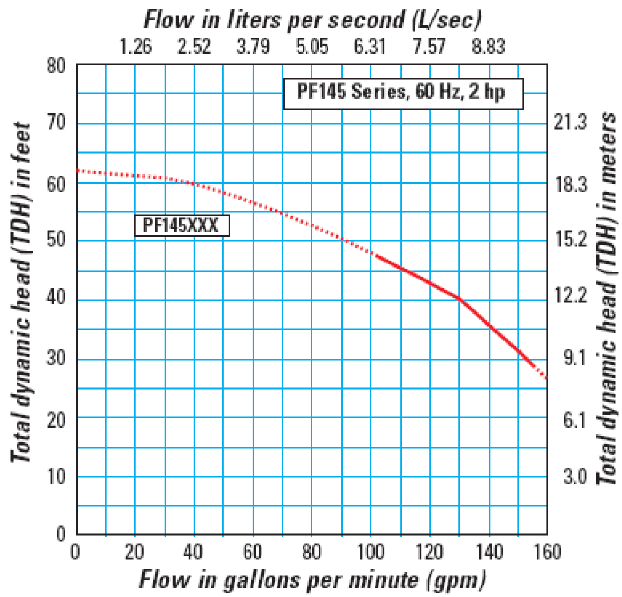
Use the results presented in the previous examples to design the aerobic treatment process using the BWF system for COD and ammonium removal. In the design, consider seasonal impacts on usage and temperature.

Solution

1. Determine the number of BWF units using an assumed ammonium loading rate of 650 g per/unit.
   a. For the present ammonium loading of 1859 g/d, the number of filters is:
      BWF units = (1859 g/d) / (650 g/filter) = 2.9; use 3 filters
   b. Under future conditions, the ammonium loading is estimated at 2417 g/d:
      BWF units = (2417 g/d) / (650 g/filter) = 3.7; an additional filter can be added in the future
2. Estimate the BWF hydraulic loading based on the expected current summer flowrate and 3 filters:
   a. BWF flow per filter= (2140 gal/d) / 3 filters = 713 gal/d
   b. Assuming a backwash flowrate of 200 gal/d for each filter (the actual value used at the Dunnigan SRRA pilot facility), the total applied flow on each filter is:
      BWF flow total= (713 gal/d) + (200 gal/d) = 913 gal/d
3. Determine the size of the BWFDT and specifications for the BWF dosing pumps and spray nozzles
   a. Tank size (minimum):
      BWFDT = ($Q_{fut,summer}$)(1 d) = 2783 gal; recommend adding 1000 gal to accommodate process operations and maintenance (O&M), for a tank size in the range of 4000 to 5000 gal
   b. Spray nozzles
      Three BWF units will have a total of 12 nozzles. The nozzles used in the BWF system are nominal 8 gal/min at 10 lb/in^2 (3/8 nozzles with 0.34-inch orifice size, HHSJX type; see Table 7-2 and Fig. 7-5).

c. Select dosing pump system:

To deliver flow at 8 gal/min and pressure of 10 lb/in^2 for 12 nozzles, a pump that can deliver a flow around 96 gal/min and with some excess pressure will be needed. The following pump curve for pump model PF145 series (pump curve from Orenco Systems shown below) will be able to provide the required flow:



The BWF units will all dose simultaneously on a 2- to 4-min cycle. For preliminary operations, a 2-min dose cycle is recommended for high performance. The dosing pump timer settings can be determined using the applied flow for each filter of 913 gal/d and the dosing frequency of 720 dose/d [(1440 min/d) / (2 min/dose)]. The pump run time for each dose cycle is:

Pump on time = (60 seconds per minute [s/min]) [(913 gal/d) / (720 dose/d)] / [(4 nozzles) (8 gal/min) = 3 s on

4. Determine the size of the BWFET and specification for the backwash pumping system

    a. Estimate the BWFET size

    The BWFET must collect enough effluent flow to backwash each filter once daily. A tank compartment with an effective volume of 1500 gal/d will be sufficient to provide the required backwash filter effluent holding volume.

    b. Backwash pumping system

    The backwash pumps need have a design value to be able to provide a flow of 60 gal/min at a pressure of approximately 25 lb/in^2.

5. Determine the size of the SSFDT and specifications for the SSF dosing system

    a. Estimate the SSFDT size

    The SSFDT is sized to accumulate the head generated by the cleaning of two filters with some additional capacity for accumulating flow for several hours for filter cleaning. A tank compartment with an effective volume of 2000 gal/d will be sufficient to provide the required holding volume.

    b. SSF dosing system

    The SSFDT contains a pump system to transfer water to the SSF. A 20-gal/min low head transfer pump system will be sufficient.

6. Compute effluent constituent concentrations for organic, particulate, and nitrogenous constituents. The data presented in Table 7-2 are used to estimate effluent quality for the BWF system.

	Process effluent, mg/L			
	UAF		BWF	
Constituent	Winter	Summer	Winter	Summer
COD	887	189	26.6	5.7
TSS	124	26	1.2	0.3
BOD	266	57	5.3	0.6
sCOD	532	113	16.0	3.4
Ammonium-N	266	266	5.3	2.7
Nitrate-N	0	0	261	263
Alkalinity	2064	2068	212	198

Comment

The values shown in this example are expected values when recycled water with high nitrate content is used to flush toilets. Because the Dunnigan SRRA pilot system has not been operated in a recycle mode at the time this manual was prepared, the guidance shown in this example is preliminary. Performance of the BWF system may vary with temperature and specific water chemistry.

7.3 OTHER CONSIDERATIONS

Several techniques can be used to keep the BWF system working at maximum performance, including the application of operation and maintenance, successful process commissioning and startup, process monitoring, and process expansion to meet increased loading needs. These items are described in this section.

7.3.1 Process Startup

To function properly, the BWF system is required to nitrify the ammonium present in the SRRA wastewater. However, nitrification can take 8 to 12 weeks to develop, depending on the season when process startup takes place. In the winter season, startup will be delayed by the cool winter temperatures. Furthermore, warmer temperatures generally observed during summer months can accelerate this reaction. The addition of 5 percent of the packing from an existing filter can be used to inoculate the filter and enhance the rate of nitrification development. In general, about three months should be allocated for process startup so that steady-state conditions can be achieved. Water recycling cannot take place until full nitrification is established. The BWF effluent will need to be diverted to an alternate dispersal area until steady-state conditions are achieved.

7.3.2 Process Monitoring

A few variables should be monitored to ensure that the BWF system is working as designed. Some parameters can be monitored continuously, while other parameters must be determined through grab sampling. The key variables for continuous monitoring are pH and turbidity.

A drop in pH is indicative of insufficient alkalinity to buffer the nitrification reaction. In the case of low pH, the chemistry should be investigated to determine the state of the carbonate and nitrogen systems. As nitrification develops, water recycling should be initiated to generate the process alkalinity that is required. Alternately, alkalinity in the form of soda ash can be added to the BWFDT at the rate required to control the pH.

Continuous monitoring of turbidity can be helpful to detect problems related to process stability. In the pilot system operated at the Dunnigan SRRA, the problems with process stability were related to (1) interruptions in the operation of the fan, which supplies oxygen to the biomass in the filter, (2) the need for the addition of more packing, or (3) the need for an additional backwash cycle to dislodge areas where the packing is consolidated.

The parameters recommended for grab sampling include those identified in the design examples. If there is a large deviation from the expected values, options for improved operation should be investigated. Suggested procedures for improved operation, i.e., improved BOD, COD, and/or ammonium removal, include increasing the dosing frequency, reducing the loading, cleaning the spray nozzles if there is evidence of fouling,and/or adding BWF units, as described below.

7.3.3 Future Expansion

Filter units can be added, if needed, to meet future demands with higher loading. If there are no issues or concerns related to meeting the treatment needs, then additional filters will not be required. If filter units are added, they should be installed so that they operate using the same dosing and backwash systems, and no supplemental pumps will be required.

7.4 OPERATION, MAINTENANCE, AND TROUBLESHOOTING

This section contains operation, maintenance, and troubleshooting information.

Under normal conditions, it is not expected that significant operation and maintenance activities will be required. However, several activities can ensure optimal performance.

Table 7-4
Operation, maintenance, and troubleshooting

Maintenance item	Frequency	Time, h	Tools/materials	Estimated annual cost, $[a]
Check spray nozzles (clean if needed)	1 mo	0.25	Water hose	300
Check to make sure that the fans are operational	1 mo	0.05	-	60
Observe a backwash cycle to confirm operations	1 mo	0.25	-	300
Check for media loss	1 mo	0.05	-	60
Confirm operation of the backwash indexing valve	1 mo	0.05	-	60
Check and adjust backwash pressure	1 mo	0.05	-	60
Add media to the recommended level	1 y	0.25	Extra packing	212.50
Clean out the BWFET if solids are present	1 y	1	Service contract	500
Check the current and voltage on the dosing and backwash pump systems	1 y	0.25	Multimeter type tester	25
Check operation of water level sensing equipment	1 y	0.5	Probe	50

a. Estimated costs include labor and materials. Labor costs estimated assuming a rate of $100/hr.

8.0 TERTIARY FILTRATION (SLOW SAND FILTER)

The slow sand filter (SSF) is a biological sand filter that is used to meet effluent turbidity standards and provide supplemental total coliform removal, as discussed in Sec. 10. Background information and procedures for design of a SSF system are presented in this section.

This section describes the approach for developing a SSF design as shown in Chart 8-1.

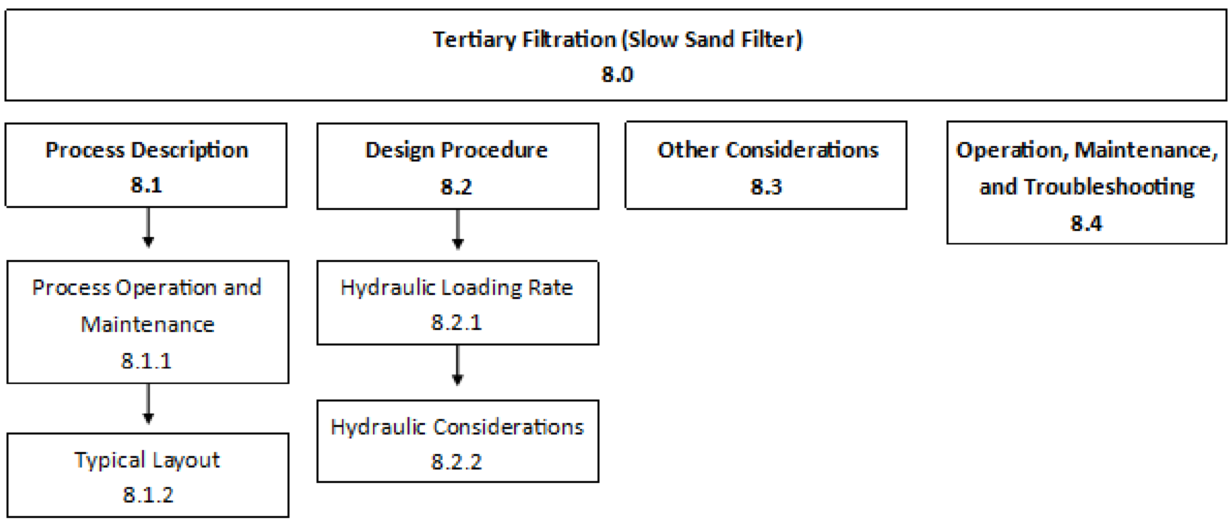


Chart 8-1
SSF design

The operation and maintenance of SSF systems, which use the oldest water treatment technology still in common use, are intuitive to operate and are relatively low cost. A diagram of the slow sand filtration process is shown on Fig. 8-1.

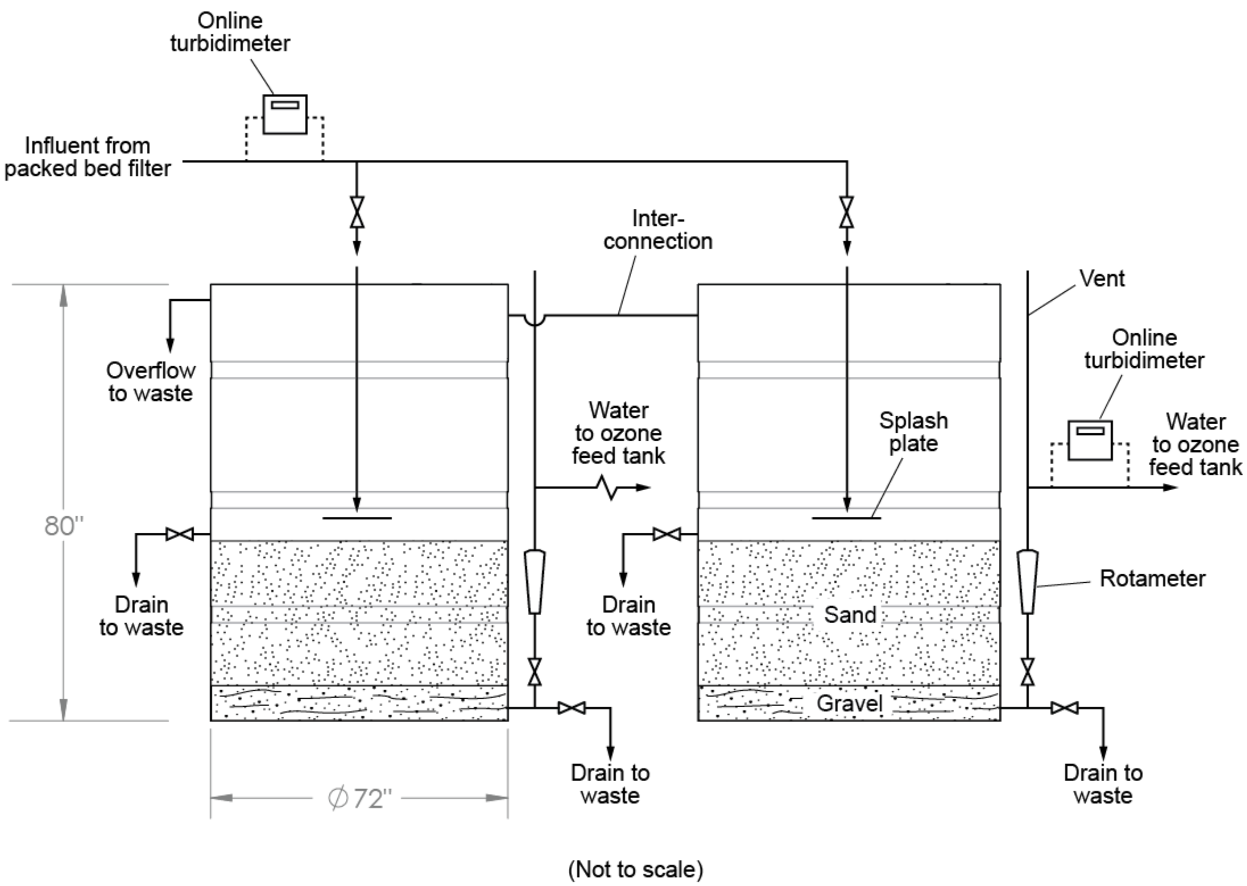


Figure 8-1
Two slow sand filters in parallel (all piping shown on diagram is 1- to 1.5 inch schedule 80 PVC; each slow sand filter system is composed of two 1,000-gal, high-density polyethylene (HDPE) tanks; filter sand; and associated controls and plumbing.)

8.1 PROCESS DESCRIPTION

Slow sand filtration removes fine particles via straining and depth filtration. A layer of organic material called the schmutzdecke forms on top of the sand media when the filter is mature (approximately six weeks). In addition, biofilm also develops within the sand bed. Biofilm development depends on the nutrient concentration and temperature of the process influent. The schmutzdecke is important for biological filtration. However, it is not important for these SSF, which are used for the removal of turbidity. The removal of the schmutzdecke during maintenance is not expected to affect the effluent quality. Filter ripening is also important for disinfection; about 2- to 3-log coliform and 4-log protozoa removal is achievable in a SSF.

8.1.1 Process Operation and Maintenance

Particles that are transported into the sand bed are removed by the biofilm via the mechanisms of depth filtration: interception, sedimentation, and diffusion. Smaller sand grain size results in better particle capture because of increased opportunities for these depth filtration mechanisms to occur. Whether a particle is removed by the biofilm depends on attachment. The number of particles that attach per number of collisions is defined as the filter α factor.

The energy of the influent flow must be dissipated via a splash plate to prevent disturbance of the schmutzdecke. As head builds up in the sand bed, the water level in the tank will rise and the splash plate will become submerged. In this case, the influent flow is dissipated by the head over the filter.

Water is maintained above the sand bed at all times by the invert of the effluent pipe. The depth of the sand bed is 2 to 4 ft and is supported by a layer of gravel, which maintains flow to the effluent pipe. An overflow is located approximately 3 ft above the invert and allows head to build up over the sand bed. A sight gauge or water level sensor is used to monitor filter clogging. Influent and effluent turbidity are monitored continuously via online turbidimeters, as described in Sec. 13.

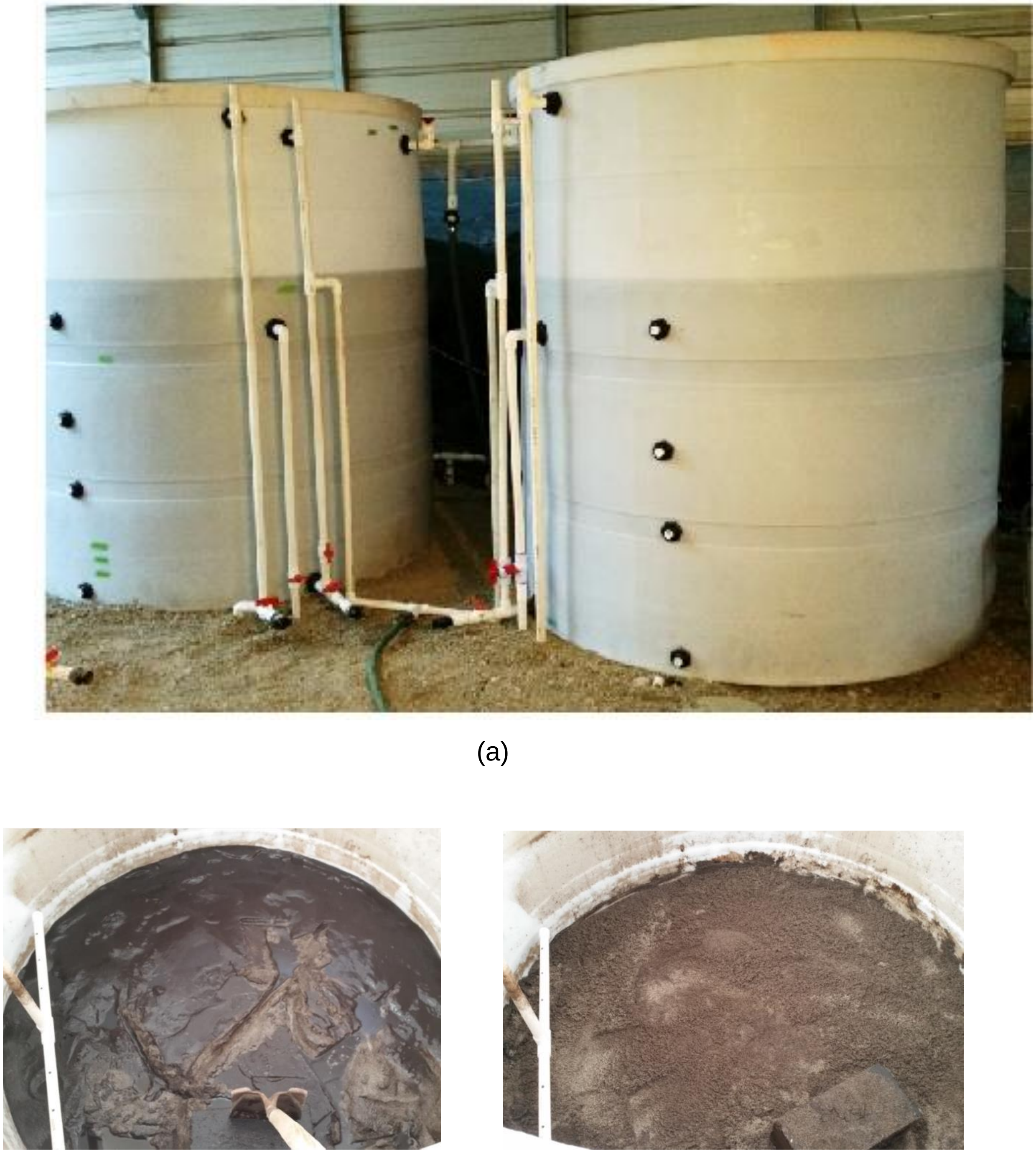
Periodically, clogging of the filter surface will reduce the rate of filtration through the sand bed and will require the sand filter surface to be cleaned to restore the filtration rate. The SSFs installed at the Dunnigan SRRA pilot facility are shown on Fig. 8-2. For SSF systems, the time interval between maintenance depends on how rapidly clogging develops in the schmutzdecke. A shutoff valve is used on the influent pipe to each filter so the filter can be taken offline for maintenance. A filter drain just above the sand bed is opened to decant the water above the sand. The filter can be drained below the top of the sand bed via a lower drain at the gravel bed. During maintenance, the water level is either lowered below the top of the sand bed and the schmutzdecke is manually scraped and discarded, or the water level is maintained above the sand bed and the top of sand bed is removed using a vacuum truck. A SSF before cleaning with the water drained from above the sand, and after manual scraping, is shown on Figs. 8-2b and 8-2c, respectively.

To refill the SSF after maintenance, potable or filtered water is applied from the bottom of the filter such that any air trapped in the sand bed is purged from the pores as water flows up through the sand. A hose bib or cam lock fitting on the filter underdrain is a convenient location for backfilling the filters. The filters should be refilled to the invert water level.

8.1.2 Typical Layout

The SSFs are arranged in parallel with a common influent line so that all filters maintain the same amount of head. The number of SSFs needed is determined by the design procedure described below. The SSF should be constructed using a tank that will reduce incident solar radiation (UV) to minimize algae growth. Various materials are suitable for tank construction. Common tank materials include polyethylene, epoxy lined metal, and concrete. Each provides a different benefit. For example, polyethylene is low cost, epoxy-lined metal is commonly used at Caltrans sites, and concrete has been used historically, but no one material is necessarily better than another. Placing the filters under a canopy can be advantageous for operation and maintenance activities and to extend the life of the equipment.

Access to the SSF should be provided for operation, maintenance, and water quality testing. Vehicle access must be provided to allow sand media to be replaced annually. Sand can be added to the filters using a loader or forklift. Operators must have access into each filter for cleaning and other maintenance activities in and around the filters. A potable water source with adequate pressure should be supplied near the SSFs for cleaning operations and filter backfilling. If filtered water will be used for filter backfilling, a return line from the product water pressure tank should be equipped with a hose bib and labeled appropriately.



(a)

(b) (c)

Figure 8-2
SSF installed at the Dunnigan SRRA pilot facility:
(a) SSF with No. 30 sand (left) and No. 20 sand (right),
(b) SSF with head drained prior to cleaning, and
(c) SSF after cleaning and before placing back into service
(Water is added from the bottom before flow is resumed.)

8.2 DESIGN PROCEDURE

The design of SSFs is based on several hydraulic considerations and selection of appropriate filter media. The parameters for design are discussed below.

8.2.1 Hydraulic Loading Rate

The SSF filtration velocity, or hydraulic loading rate (HLR), can range from 0.016 to 0.16 gallons per minute per square foot (gal/min·ft^2). Based on pilot testing, a value of 0.025 min·ft^2 is a typical flowrate for preliminary sizing of SSFs. The SSFs installed at the Dunnigan SRRA pilot facility were round tanks with a diameter of 6 ft and surface area of 28 ft^2. The flowrate through the pilot SSFs ranged from about 1.0 gal/min at startup to 0.25 gal/min prior to maintenance. The flow used for sizing is the average future summer flow; however, it is an option to install enough SSFs to meet the current demand and add filters later as needed.

8.2.2 Hydraulic Considerations

Incremental headloss occurs because of deposits or biofilm development within the sand bed. Most of this headloss occurs through the schmutzdecke. The buildup of pressure head on two SSFs, as observed in at the Dunnigan SRRA pilot facility is shown on Fig. 8-3. The larger No. 20 sand can process more flow than the No. 30 sand, while both sand sizes provided similar performance for turbidity removal. Therefore, No. 20 sand is recommended for the best balance of extended filter runs and performance.

The filtration rate was found to become somewhat erratic as the flowrate was reduced. In some cases, gas bubbles were observed coming from the filter bed as the filters clogged. The gas bubbles are interpreted to be nitrogen gas produced by denitrification reactions in the sand bed because the gas bubbles seemed to be related to the reduced flow, which allowed anoxic conditions to develop. The gas bubbles in the sand bed have a negative impact because they interfere with filtration because of an effect known as gas binding. Therefore, the filters should be cleaned prior to development of denitrification in the sand bed, i.e., low filtration rates.

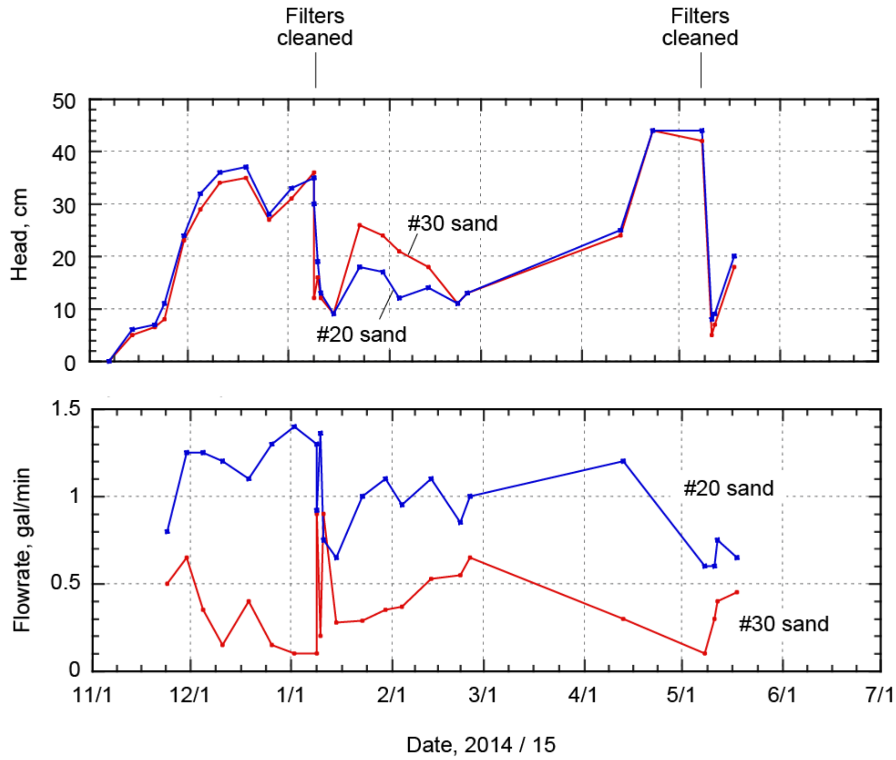


Figure 8-3
Head accumulation and flowrate for pilot SSFs

Sand Specifications

Coarse sands with mesh size No. 20 or No. 30 (ASTM C 778) appear to be the best sands to use for the tertiary filtration of SRRA wastewater as described above. A sample particle size distribution of No. 20 and No. 30 sands (SRI Supreme, Marysville, CA) is given on Fig. 8-4. Effective size (ES) is a critical parameter in specifying filter sand; it is defined as the particle size at which 90 percent of the sand is retained on the screen and 10 percent passes through it (d_{10}). For example, in No. 20 sand, the d_{10} value is <0.85 mm, meaning that the d_{10} of No. 20 sand is not necessarily No. 20 mesh. A uniformity coefficient (UC), defined as the ratio of particle size correlated with 60 percent passing through a screen (d_{60}) to the effective size (d_{60}/d_{10}), between 1.5 and 4, is used to ensure adequate porosity. The sand must be washed with less than 4 percent fines passing No. 100 mesh screen (Eliasson, 2002).

The gravel underdrain and support layer consists of three to five graded gravels ranging from about 3 to 60 mm in diameter. The bottom layer contains the largest gravel size, with subsequent layers having progressively smaller gravel sizes range and each layer, ranging from 4 to 6 inches each (Hendricks, 2011). The layers for the Dunnigan SRRA pilot SSFs were 3/4, 1/2, 1/4, and 1/8 inch, with filter sand placed above the 1/8-inch layer.

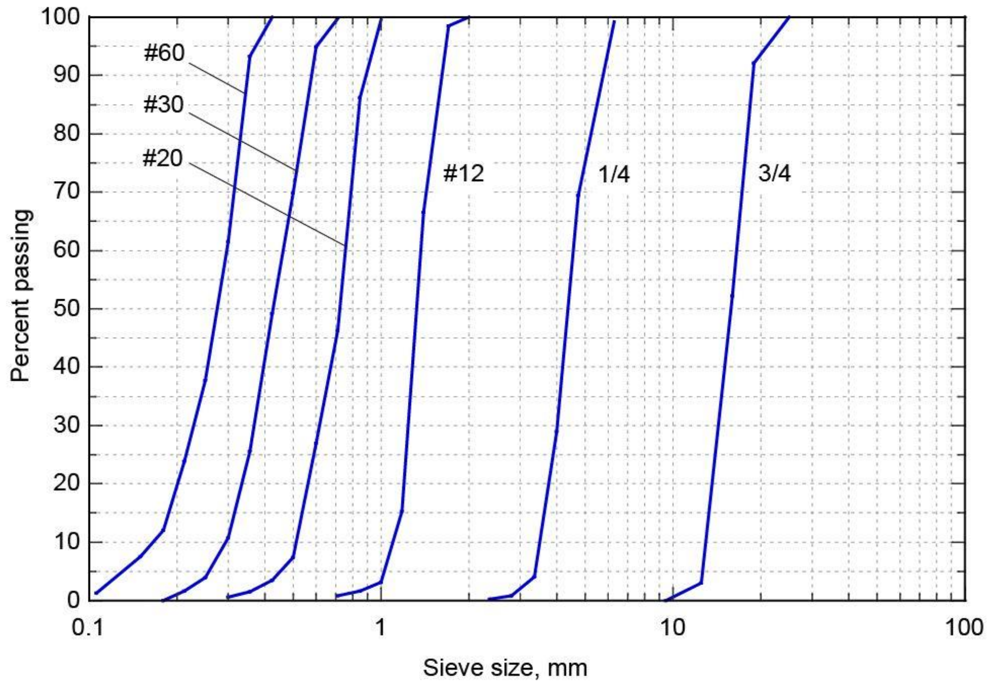


Figure adapted from SRI Supreme, Marysville, CA

Figure 8-4
Particle-size distribution for sands

The design procedure for tertiary filtration design is summarized on Chart 8-2, and demonstrated in Ex. 7.

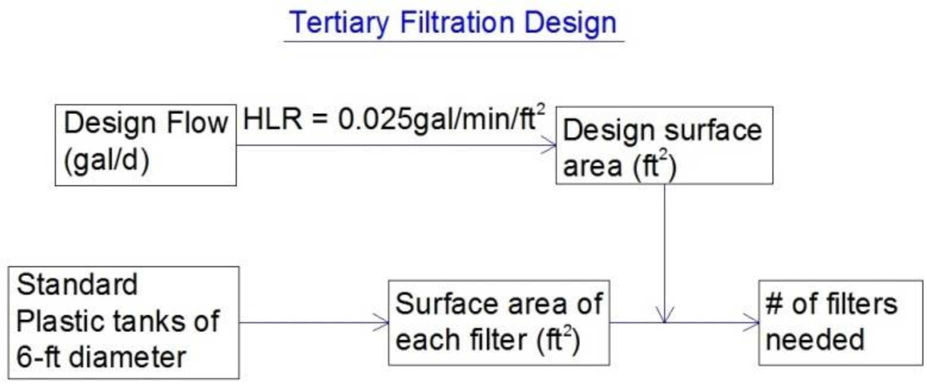


Chart 8-2
Tertiary filtration design flow chart

Example 7: Design the SSFs

Use the flow data developed in Ex. 1 and a design HLR of 0.025 gal/min·ft^2 to determine the number of SSFs required at the SRRA. Assume the filters are -6-ft-diameter black polyethylene plastic tanks. Determine the time to maintenance if the headloss through the filter increases 3/4 inch per week.

Solution

1. Peak day future flows

   a. Summary of flows developed in Ex. 1

Scenario	Unit	Present	Future
Summer	gal/d	2140	2783
Winter	gal/d	1591	2068
Mean	gal/d	1845	2399
Peak	gal/d	3690	4798

   b. The design flow based on the future summer flows is expected to be 2783 gal/d.

2. Assuming full diurnal equalization, compute the loading on the SSFs for the design flow.

   If the flow is fully equalized, the constant flow to the SSFs:

   Flowrate = (2783 gal/d) / (1440 min/d) = 1.93 gal/min

3. Compute the surface area per filter

   Filter area = $\pi r^2 = \pi 3^2 = 28.3\ ft^2$

4. Determine the number of filters that are required.

   a. Given HLR = 0.025 (gal/min)/ft^2

   b. Total filter area (ft^2) = (2 gal/min) / (0.025 gal/min·ft^2) = 77.3 ft^2

   c. Determine the number of SSFs by dividing by area per filter

      Number of filters = (80 ft^2) / (28 ft^2) = 3 filters

      Note, consider 1 additional filter to allow for flexibility in filter maintenance.

5. Filters should be constructed with 2.5 ft of freeboard from invert to overflow, to allow the filter to operate for approximately 40 weeks (wk), or 10 mo. In reality, the amount of flow that passes through the filters will depend on the performance of the upstream treatment system. At the Dunnigan SRRA pilot facility, filter cleaning was initiated when the filters began to overflow, about 4 to 6 mo after cleaning. Cleaning every 3 mo is recommended as a precaution.

Comment

While the filter can be operated to the point of overflow, the rate of headloss may not be linear, as described in the problem statement. It is more likely that the rate of flow through the filter will slow to a point where head builds rapidly, as observed after the first filter cleaning in Fig. 8-3. It is advisable to monitor the flowrate and headloss through the filters regularly to determine an appropriate maintenance trigger.

8.3 OTHER CONSIDERATIONS

At the Dunnigan SRRA pilot facility, it was found that the SSF units required limited operation and maintenance, always produced an effluent that had a low turbidity, and were predictable. Monitoring of SSF operation can be accomplished using online turbidimeters, as shown on Fig. 8-5. However, the turbidity of the filter effluent never exceeded about 1.5 NTU.

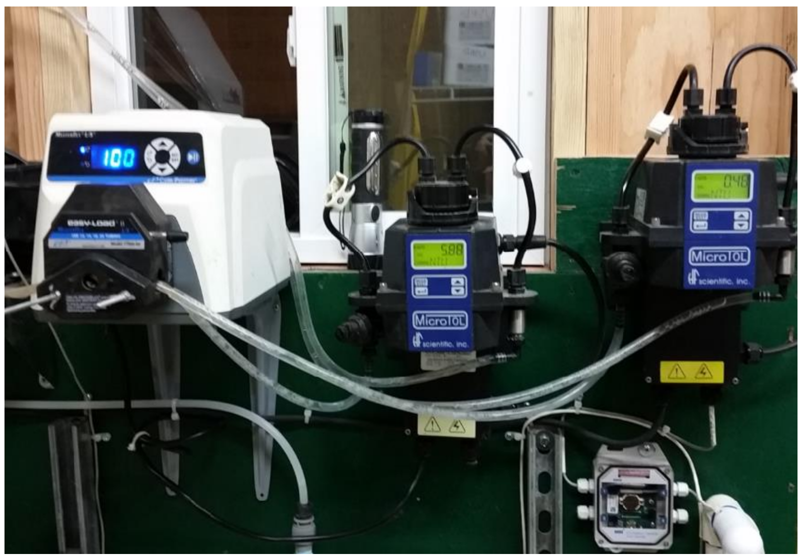


Figure 8-5
Online turbidimeters used to monitor turbidity into and out of the SSF system

In the pilot testing, it was found that the solids penetrated deeper into the filter bed of the No. 20 sand, compared to the No. 30 sand, but that about twice as much water could be filtered through the No. 20 sand prior to cleaning. Ultimately, about 1 inch of sand was removed from the No. 20 sand filter (compared to about 0.5 inch for the No. 30 sand). If the filters area cleaned about three times per year, the depth of the filter will be reduced by about 3 inches per year. It is recommended that new sand be added every year or two to replenish the full depth of the filter bed. Historically, the sand that was removed from the filter bed was washed and returned to the filter; however, for the small filter size used at SRRAs and the ready availability of sand, it will probably be more desirable to purchase new sand rather than recycling old sand. Therefore, management of the removed sand may be limited to disposal as the sand could contain protozoan pathogens and should be handled appropriately.

8.4 OPERATION, MAINTENANCE, AND TROUBLESHOOTING

The anticipated maintenance requirements include scraping and removing sand from the filter surface, washing filter surface sand, moving sand to and from storage, rebuilding the sand bed, checking sand filter effluent flowrate (rotameter), and checking and calibrating the turbidimeter.

Table 8-1
Operation, maintenance, and troubleshooting

Maintenance item	Frequency	Time, h	Tools / materials	Estimated annual cost, $[a]
Clean filter	6 mo	4	Shovel or vacuum truck	1400
Check flowrate	Continuous	-	SCADA	-
Add makeup sand	1 to 2 y	2	Sand	600
Calibrate turbidimeter	Monthly	2	Calibration standards	2600
Record head levels	Continuous	-	SCADA	-

a. Estimated costs include labor and materials. Labor costs estimated assuming a rate of $100/hr.

9.0 COLOR REMOVAL AND DISINFECTION (OZONATION)

Ozone is an extremely reactive gas that can be used as a disinfectant or an oxidant to improve water quality. It serves as a disinfectant by reacting with and rupturing the cell walls of pathogens, also known as cell lysis (Tchobanoglous et al., 2014). Ozone can also be used as an oxidant to breakdown organics in wastewater that contribute to color and odor. Ozone can be generated by the use of electrolysis, photochemical reaction, and radiochemical reaction by electrical discharge. Electrical discharge, specifically corona discharge, is the typical method used to produce ozone for wastewater treatment.

This section describes the approach for developing an ozonation system design, as shown in Chart 9-1.

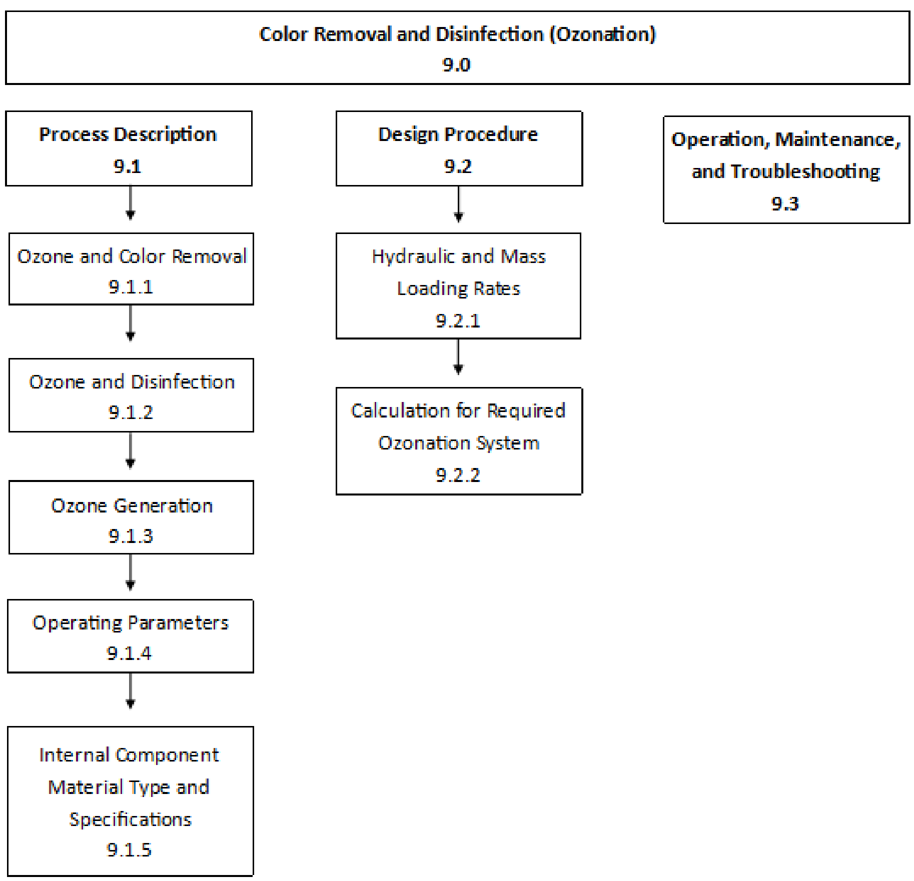


Chart 9-1
Ozonation system design

A description of the ozonation process, an ozone system design procedure, and estimates for operation and maintenance are presented in this section. The ozone system is composed of an electrically powered oxygen concentrator, ozone generator, venture injector, and contact reactor vessel, as shown on Fig. 9-1. Ozonation systems comprise multiple individual components. Together the footprint of the entire system is about 60 ft^2. However, components are generally separated to optimize space. For example, the ozone generator can be mounted to a wall inside a storage shed for protection from the environment. Ozone contact tanks generally range from about 10 to 50 gal, and dosing tanks generally range from 100 to 150 gal for average SRRA conditions. Tank sizing depends on site specific flow conditions and can be computed as demonstrated in Ex. 8.

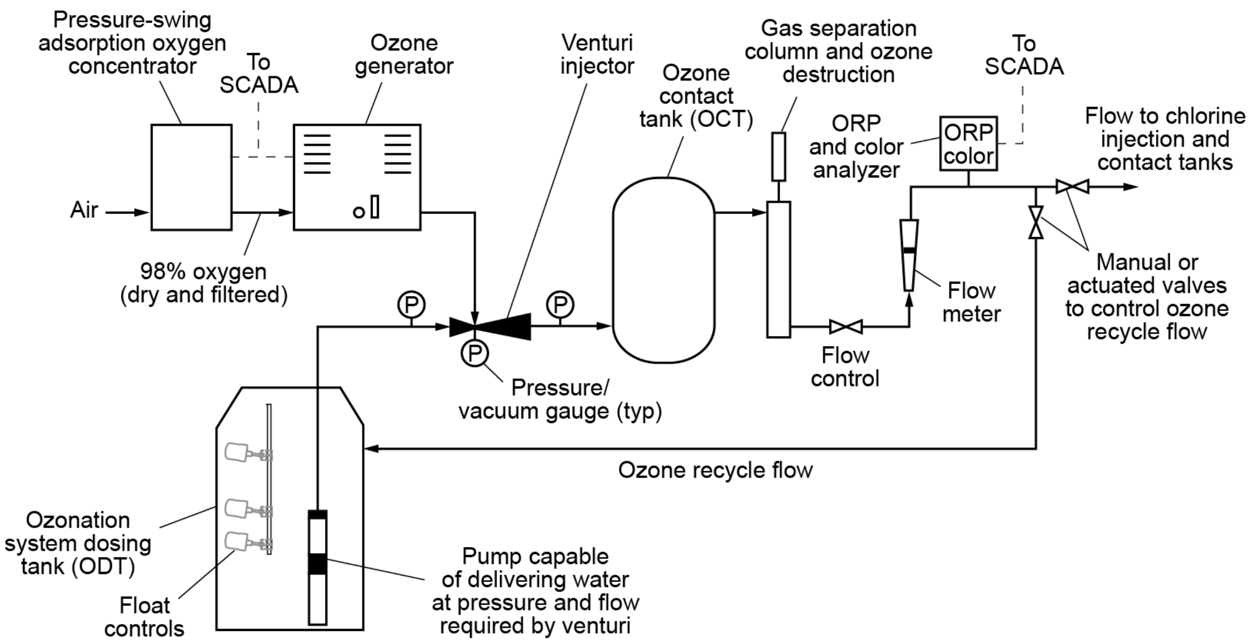


Figure 9-1
Process diagram for ozonation system
(Note that all liquid piping should be 1 inch schedule 80 PVC
and all oxygen and ozone gas tubing is 0.25-inch teflon. All pumps and ozone-generating components require power.)

9.1 PROCESS DESCRIPTION

Ozone reacts with organic compounds, including pathogens, in water in two ways: (1) direct oxidation, and (2) the production of hydroxyl radicals, which have great oxidizing powers. When ozone is introduced to water, hydroxyl and other radicals are formed via the following equations.

$$O_3 + H_2O \rightarrow HO_3^+ + OH^- \quad (9\text{-}1)$$

$$HO_3^+ + OH^- \rightarrow 2HO_2 \quad (9\text{-}2)$$

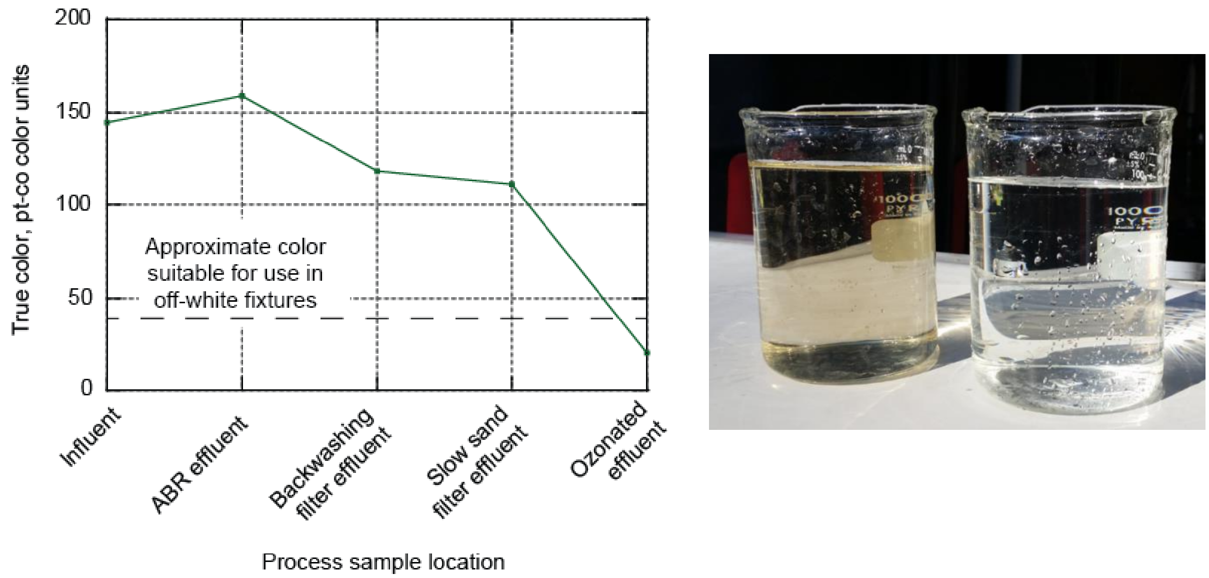
$$O_3 + HO_2 \rightarrow HO\bullet + 2O_2 \quad (9\text{-}3)$$

$$HO\bullet + HO_2 \rightarrow H_2O + O_2 \quad (9\text{-}4)$$

However, the formation of radicals is favored only at high pH values (pH ≈ 11). Thus, for most scenarios, direct oxidation with ozone is the primary mechanism for oxidation (Crittenden et al., 2005).

9.1.1 Ozone and Color Removal

Color removal is necessary to achieve disinfection and an aesthetically acceptable final effluent color for use in toilet flushing. Color can be measured as true or apparent color, where true color is measured on a filtered sample (turbidity removed) and apparent color is measured on an unfiltered sample. For the context of this section, color values refer to true color. Color is composed of the light absorbed between 400 and 800 nm as well as that fluoresced between 200 and 400 nm. An example of color observations across multiple processes in the Dunnigan SRRA pilot treatment train is shown on Fig. 9-2.

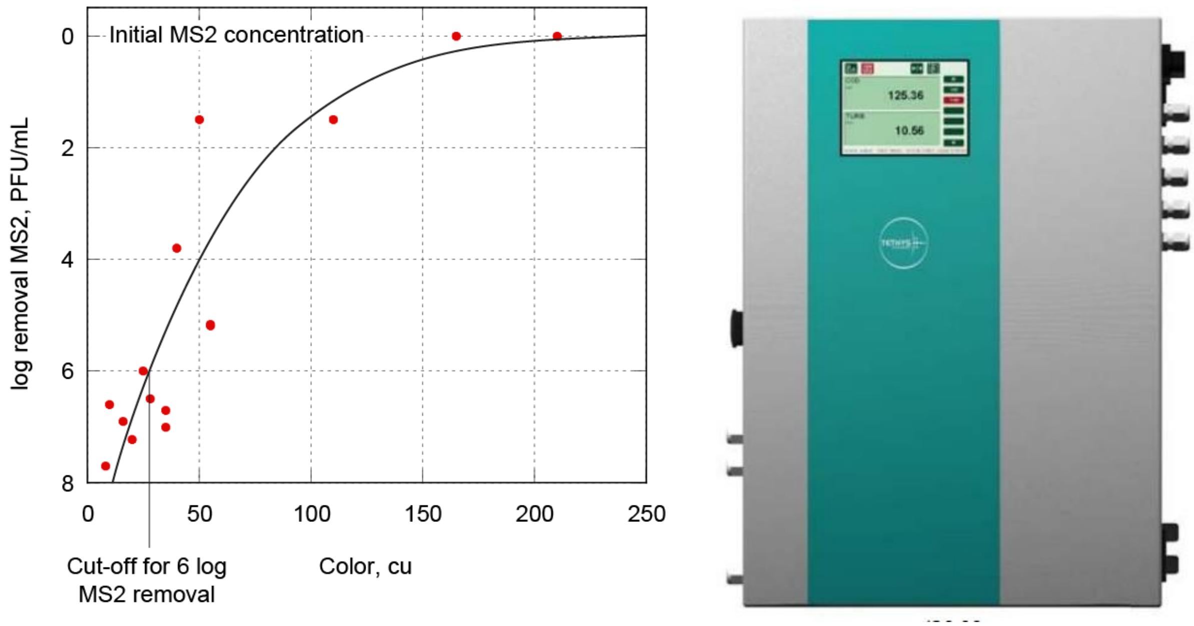


(a) (b)

Figure 9-2
Color removal with ozone at the Dunnigan SRRA pilot facility:
(a) incremental color removal through treatment steps, and
(b) water sample after SSF (left) and ozonation (right)

9.1.2 Ozone and Disinfection

To meet the definition of disinfected tertiary recycled water in CCR Title 22, disinfection, when combined with the filtration process, must also provide 99.999 percent (5-log) inactivation or removal of the plaque forming units (PFUs) of F-specific bacteriophage MS2. The ozonation process was evaluated to determine disinfection potential and to demonstrate the required bacteriophage removal during pilot testing. The ozonation process was shown to remove over 6-log of bacteriophage when the color was removed below about 30 cu. The correlation between residual color after ozonation and MS2 virus survival is summarized on Fig. 9-3. An example of an online color analyzer is also shown on Fig. 9-3.



(a) (b)

Figure 9-3
Color removal in ozone disinfection: (a) relationship between residual color and MS2 virus removal at the Dunnigan SRRA pilot facility, and (b) example of online color analyzer that can be used for verification of virus kill based on the data shown on (a)

Compared to chlorine, ozone is more effective at inactivating viruses in wastewater, does not increase dissolved solids, and is not impacted by the presence of ammonia. However, while efficacy of direct oxidation with ozone is not pH dependent, ozone residuals are more stable in acidic environments. Because ozone is not being considered as a disinfectant residual, the effluent pH is not important for the purposes of disinfection with ozone, but is important for disinfection with chlorine, as discussed in Sec. 10.

9.1.3 Ozone Generation

Ozone is generated using oxygen from the oxygen concentrator. As the water flows through the Venturi injection system, it creates a vacuum that pulls ozone gas from the ozone generator and injects the ozone into the flow of water. The water then flows through the contact tank, which is pressurized to provide more effective gas transfer of ozone to the water. A degas system removes and destructs any undissolved ozone that would otherwise escape to the atmosphere.

A recycle stream of ozonated effluent to the ozone dose tank can be introduced to enhance the ozonation process. Depending on water quality, the influent flow can be recycled through the ozone system to improve color removal, enhance pathogen inactivation/destruction, and further oxidize organic compounds. For the Dunnigan SRRA pilot tests with low-flow fixtures (e.g., 1.28-gal/flush toilets, waterless urinals), recirculation of ozonated water multiple times was needed to obtain adequate color removal. The recycle rate can be adjusted via an actuated or manual gate valve on the ozone dose tank effluent line. The number of passes required is determined on system startup and commissioning, and as needed during regular operation, to obtain an ozonated effluent with a color value less than about 30 cu.

The oxygen concentrator, ozone generator, and appurtenances must be kept in a climate-controlled environment to ensure equipment longevity (see Fig. 9-4a). All internal components must be kept dry (sheltered from water, snow, humidity, etc.) and protected from excessive dust, vibration, solar radiation, and mechanical shock. Sufficient clean air unit cooling must be supplied to keep the equipment between 55 and 85 degrees F, and air intake and exhaust vents must be unobstructed. The ozone injection, contact tank, degas unit, and thermal destruction can be placed outside to reduce the possibility of ozone gas escaping in an enclosed area (see Fig. 9-4b).

(a) (b)

Figure 9-4
Dunnigan SRRA pilot facility ozone equipment: (a) oxygen concentrator and ozone generator, and (b) Venturi injector, contact tank, flow meter, ORP analyzer, degas unit, and thermal destruct unit

9.1.4 Operating Parameters

Typical values of key operating parameters for the oxygen concentrator and ozone generator are given in

Table 9-1. The Venturi injector should be sized to dissolve a minimum of 90 percent of the ozone gas into the water flow continuously.

9.1.5 Internal Component Material Type and Specifications

Because ozonated water is highly corrosive, all materials must be corrosion resistant and checked for chemical compatibility with ozone. Compatible materials where ozone contact will occur include schedule 80 PVC, kynar, teflon, 300 series stainless steel, viton, ethylene propylene diene terpolymer (EPDM), and concrete.

Table 9-1
Ozonation capacity and power rating requirement

Parameter	Unit	Value from Dunnigan SRRA pilot facility
	Oxygen concentrator	
Pressure	lb/in^2	17
Oxygen output	ft^3/h	17
Dewpoint	°F	-100
Oxygen purity	%	90 +/- 5
Feed air requirements	ft^3/min	2.5
Power	kW	0.6
	Ozone generator	
Ozone concentration	%	1- 10 (by weight, adjustable)
Output	g/h	25
Oxygen requirement	ft^3/h	12
Working pressure	lb/in^2	20-120
Power	kW	0.48
	Venturi injector	
Water flowrate	gal/min	10
Water inlet pressure	lb/in^2	35 – 40
Water vacuum pressure	in Hg	-10

The oxygen concentrator internal components include cycle pressure gauge, absorbers (molecular sieve), pressure regulator, air flow meter, capacitor, circuit board, terminal strip, valve block, air compressor, air intake, cooling fan, heat exchanger, equalization valve, mixing tank, exhaust muffler and intake resonator. Ambient air is drawn into the oxygen concentrator through the gross particle filter and feed air compressor. A mix tank is provided downstream of the absorbers and the process is controlled by feed, equalization, and waste solenoid valves as well as the pressure regulator.

The corona discharge-type ozone generator internal components include a potentiometer, ozone flow control valve, and water drain system to prevent the ozone generator from flooding. The ozone flow control valve is located downstream of the ozone generator to maintain working pressure across the ozone cell (e.g., 20-120 lb/in^2).

9.2 DESIGN PROCEDURE

Effective ozonation is mainly dependent on two aspects of system design: (1) the mass transfer device used to move ozone into the water (e.g., Venturi injector), and (2) the contact chamber where the reactions take place. In the past, mass transfer was maximized by introducing ozone into deep basins with porous diffusers. Modern high-concentration ozone generators with high-efficiency pressurized contactors and de-gassing systems allow for more compact ozonation systems. Because the technology is progressing quickly, it is recommended to consult with an ozonation system vendor for specific product details and specifications before purchasing an ozone system.

9.2.1 Hydraulic and Mass Loading Rates

The ozone demand for a system is proportional to the amount of oxidizable material in the system. Ozonation is a chemical reaction; thus, the inorganic and organic substances in the water will require a certain amount of ozone to be oxidized. Inorganic constituents of concern include manganese, iron, and hydrogen sulfide; for complete oxidation, these constituents require 0.44, 0.88, and 3 milligrams (mg) of ozone per mg of constituent, respectively.

The removal of organic constituents with ozone is much harder to predict, because many types of organic species such as carboxylic acids, ketones, and aldehydes are not reactive with ozone. In addition, even if a species is reactive with ozone, it may decompose to species that do not react with ozone. Thus, the best method to predict ozone demand is pilot testing; however, an indirect method of determining the ozone demand for organic species in water is by determining the COD of the water. The COD is a colorimetric laboratory test that determines the oxygen needed to convert all carbon containing species in the water to carbon dioxide. If all the organic species in the water were sensitive to ozone, then it would require 2.5 mg of ozone to remove 1 mg of COD.

Examples of process set points for the oxygen concentrator and ozone generator are a flowrate of 9 gal/min, an ozone flowrate of 12 ft^3/h at 10 lb/in^2, a Venturi inlet pressure of 40 lb/in^2, and a Venturi vacuum pressure of -10 inches Hg.

9.2.2 Calculation for Required Ozonation System

Ozone demand from organic and inorganic species is part of the instantaneous ozone demand typically, while disinfection is carried out by the ozone residual, or ozone remaining after the instantaneous demand has been met. The disinfection efficacy is dependent on the type of organism and the product of residual concentration and contact time (C_RT).

Because ozone levels decay over time, monitoring of ozone concentration in the contact tank is recommended when determining ozone residual available for disinfection. The amount of a particular organism that is then killed or inactivated is generally given in terms of log reduction or log removal. Log reduction refers to the log (base 10) number organisms removed and can also be given as percent removal. For example, 99.9 percent removal of a microorganism is equivalent to 3-log removal. The C_RT required for a given log removal of various organisms is provided in Table 9-2.

While bacteria and virus removal has been demonstrated through challenge testing and coliform measurements, protozoan pathogens (esp. Cryptosporidium) are difficult to remove with ozone. However, the SSF described in Sec. 8 is known to remove protozoan organisms effectively.

Table 9-2
C_RT values for various organisms at pH 7

Organism	C_RT, mg-min/L	Log reduction
Virus	0.5 – 0.6	4
Bacteria	0.02 - 0.04	4
Protozoa	12 - 13	3

Source: Tchobanoglous et al., 2014

The design procedure for a color removal and disinfection (ozonation) system is summarized on Chart 9-2, and demonstrated in Ex. 8.

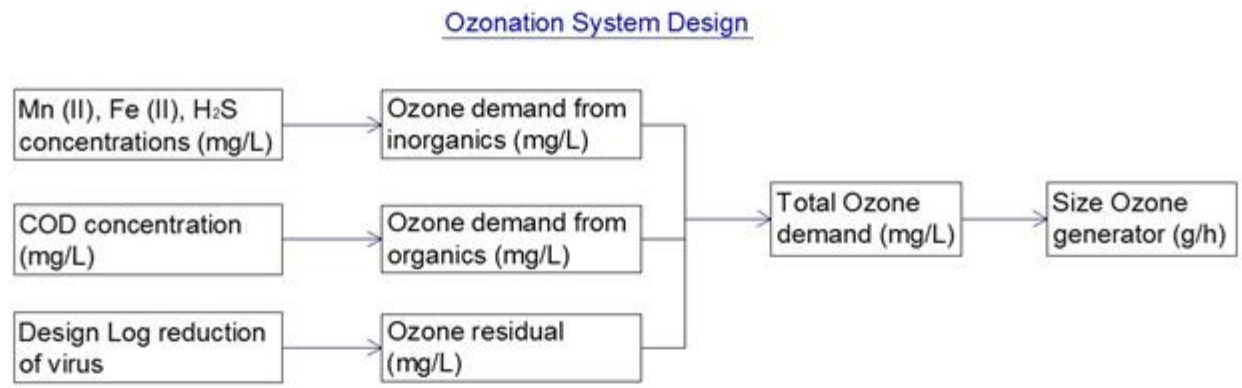


Chart 9-2
Ozonation system design flow chart

Example 8: Remove color and disinfect with ozone.

Estimate the ozone demand for influent water with the following characteristics, which are characteristic of SSF process effluent, and a contact time of 1 min:

- Fe(II) = 0 mg/L
- Mn(II) = 0 mg/L
- H_2S = 0 mg/L
- COD = 27 mg/L
- Required 5-log reduction of viruses

Also size the ozone generator and comment on the likelihood of effective color and virus removal.

Solution

1. Determine ozone demand from inorganic species:

   Inorganic ozone demand, mg/L =

   0.44 mg/L [Mn(II) mg/L] + 0.88 mg/L [Fe(II) mg/L] + 3 mg/L (H_2S mg/L) =

   0.44 mg/L (0 mg/L) + 0.88 mg/L (0 mg/L) + 3 mg/L (0 mg/L) = 0 mg/L

2. Determine ozone demand from organic species:

Organic ozone demand, mg/L = 2.5 mg/L (COD mg/L) = (2.5) (27) = 66 mg/L

3. Determine the residual ozone needed for disinfection:

   C_RT for a 4-log reduction of viruses with ozone = 0.6 mg-min/L

   Ozone residual = C_RT/time = (0.6 mg-min/L) / (1 min) = 0.6 mg/L

4. Add inorganic, organic, and disinfection ozone demand

   Ozone demand = inorganic + organic + disinfection

   = 0 mg/L + 66 mg/L + 0.6 mg/L = 67 mg/L

5. Size the ozone generator

   For a single-pass system, the required ozone output is:

   Ozone output = (10 gal/min) (3.785 L/gal) (67 mg/L) (60 min/h) / (1000 mg / g)= 152 g/h

   For an ozone transfer efficiency of 75 percent, the required ozone production is:

   Ozone generator = (152 g/h) / (0.75) = 203 g/h

6. Comment on virus removal

   It was demonstrated at the Dunnigan SRRA pilot facility that, if the ozone demand for the color was met, virus removal exceeding 5-log was ensured. Sizing the ozone system for COD oxidation will exceed the requirements for color removal and will therefore achieve the required level of disinfection.

Comment

The ozonation system will consist of an oxygen concentrator, 10-gal contact tank, 10-gal/min Venturi injector, and two 160-gram-per-hour (g/h) ozone generators operating in parallel. Note that the proposed ozone generator strategy is designed for two generators to run at partial capacity in parallel and will also allow the system to be run with one generator out of service. The ozone system will need a dosing pump capable of providing a flow of 10 gal/min at about 50 lb/in^2. The pump will operate on demand for about 4 hours per day (h/d) in a single-pass mode. Alternately, a 30-g/h generator could be operated continuously in a multi-pass mode. The multi-pass mode of operation was validated at the Dunnigan SRRA pilot facility. The oxygen concentrator will need to be determined on the basis of the type of ozone generator that is selected.

9.3 OPERATION, MAINTENANCE, AND TROUBLESHOOTING

It is recommended that an inventory of spare parts be maintained onsite in the event of a component failure. During the Dunnigan SRRA pilot study, components related to the ozone system that failed included float switches, dosing pump, motor contactor, seals on oxygen concentrator air compressor, drive board and transformer on ozone generator, fuses, and gas delivery tubing (HDPE tubing was used rather than teflon). This section contains operation, maintenance, and troubleshooting information.

Table 9-3
Operation, maintenance, and troubleshooting

Maintenance item	Frequency	Time, h	Tools / materials	Estimated annual cost, $[a]
Rebuild air compressor	Annual	2	seal kit	250
Make miscellaneous ozone repairs	As needed	8	varies	1000
Clean filters	Monthly	0.5	none	600
Check Venturi injection pressure	Monthly	0.25	Read pressure gauge	300
Calibrate online ORP and color analyzers	Monthly	0.5	Field instruments to check online measurement	600
Check for signs of corrosion or gas leaks	Monthly	0.25	Visual check	300

a. Estimated costs include labor and materials. Labor costs estimated assuming a rate of $100/hr.

Advantages to using ozonation include:

- Ozone is effective over a wide pH range and rapidly reacts with bacteria, viruses, and protozoans and has stronger germicidal properties then chlorination. Ozone has a very strong oxidizing power with a short reaction time.
- The treatment process does not add chemicals to the water.
- Ozone can eliminate a wide variety of inorganic, organic and microbiological problems and taste and odor problems. The microbiological agents include bacteria, viruses, and protozoans (such as Giardia and Cryptosporidium).

Disadvantages to using ozonation include:

- There are higher equipment and operational costs and it may be more difficult to find professionals proficient in ozone treatment and system maintenance.
- Ozonation provides no germicidal or disinfection residual to inhibit or prevent regrowth.
- Ozonation by-products are still being evaluated and it is possible that some by-products may be carcinogenic. These may include brominated by-products, aldehydes, ketones, and carboxylic acids. This is one reason that the post-filtration system may include an activated carbon filter.
- The system may require pretreatment for hardness reduction or the addition of polyphosphate to prevent the formation of carbonate scale.
- Ozone is less soluble in water, compared to chlorine, and, therefore, special mixing techniques are needed.
- Potential fire hazards and toxicity issues can be associated with ozone generation.

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10.0 RESIDUAL DISINFECTION (CHLORINATION)

Chlorination is the most common method to disinfect pathogens in water and wastewater and is also used to control biological growth and biofouling in the associated piping infrastructure. In drinking water, chlorine is used commonly as an oxidant to control iron and manganese formation in groundwater supplies and to break down organics to improve taste, odor, and color.

This section describes the approach for developing a chlorination system design, as shown in Chart 10-1.

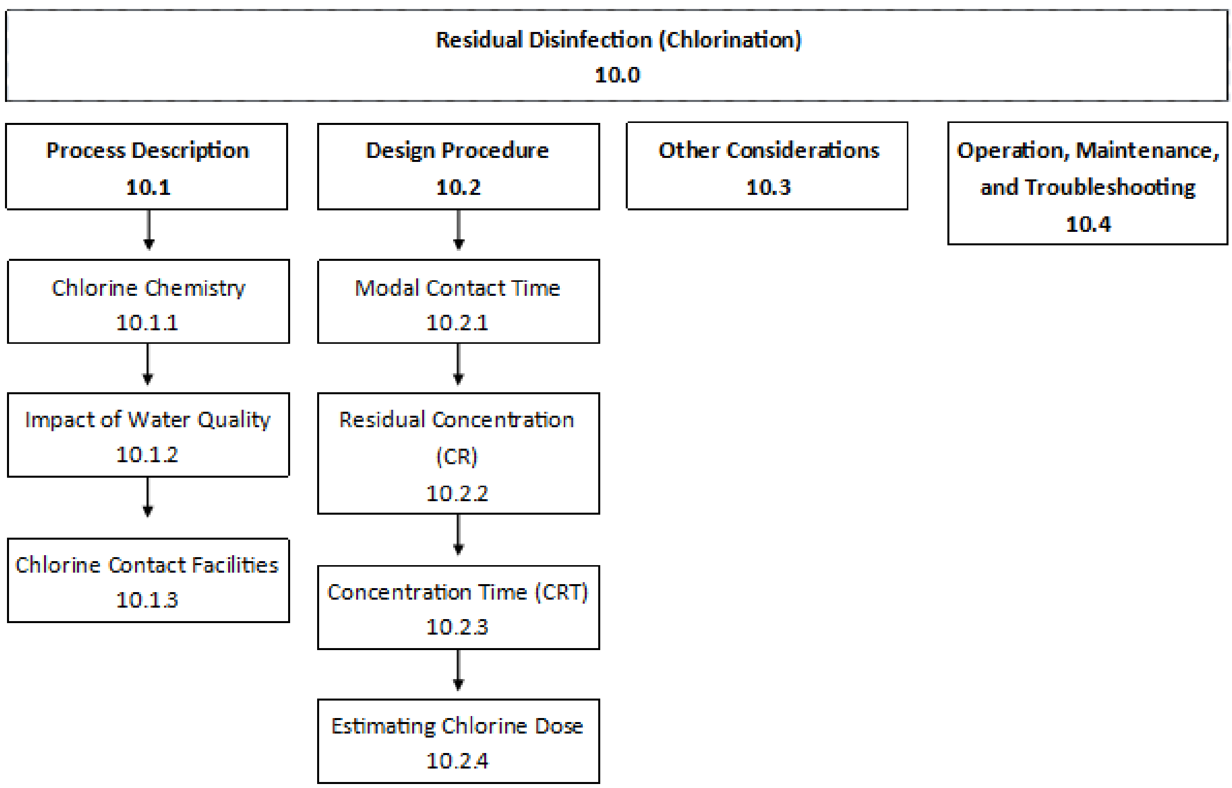


Chart 10-1
Chlorination system design

The chlorination system shown on Fig. 10-1 is presented for purposes of the following discussion. To understand the chlorine disinfection process, it is useful to review the chlorine chemistry, a design procedure, and operation and maintenance needs presented in this section. Each chlorine contact tank is typically about 250 to 300 gal in volume. The tank at the Dunnigan SRRA site is 4 ft in diameter and the entire system has a footprint of approximately 60 ft^2.

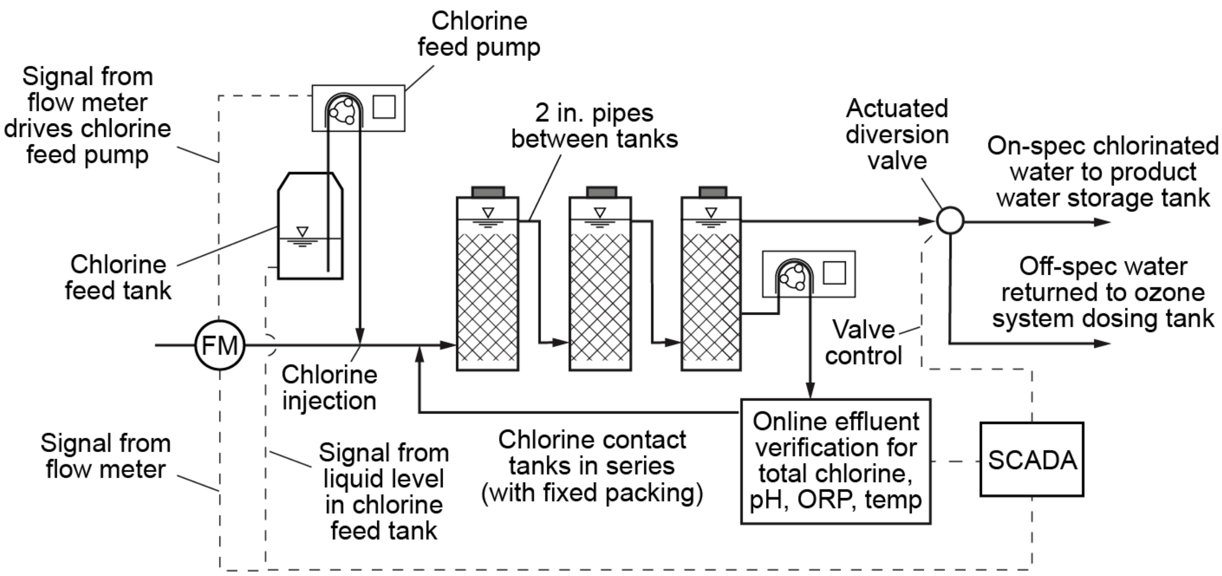


Figure 10-1
Chlorination system diagram showing tank in series for an extended contact time
(Note that all piping in this configuration, except for transfer pipe between contact tanks, is 1- to 1.5-in. schedule 80 PVC.)

10.1 PROCESS DESCRIPTION

Chlorine can be applied to water for disinfection in many forms, including chlorine gas (Cl_2), reacted with water in the absence of ammonium (hypochlorous acid [HOCl] and hypochlorite [OCl^-]), and reacted with water in the presence of ammonium as chloramines (monochloramine [NH_2Cl], dichloramine [$NHCl_2$], and nitrogen trichloride [NCl_3]).

Because of the hazards associated with transporting and storing chlorine gas, many wastewater treatment facilities use chlorine in an aqueous solution of water and sodium hypochlorite (NaOCl). Sodium hypochlorite is most effective at pH values below 7.5 to favor the formation of HOCl rather than OCl^-. As shown on Fig. 10-2, above a pH of 7.5, OCl^- becomes the dominant species in solution, which is important because HOCl is a more effective disinfectant than OCl^-. For example, HOCl is about 100 times more effective at killing *E. coli* than OCl.

The main equations governing the formation of HOCl are:

$$NaOCl + H_2O \rightarrow HOCl + NaOH \qquad (10\text{-}1)$$

$$HOCl \rightarrow H^+ + OCl^- \quad \left(K = 10^{-7.5}\right) \qquad (10\text{-}2)$$

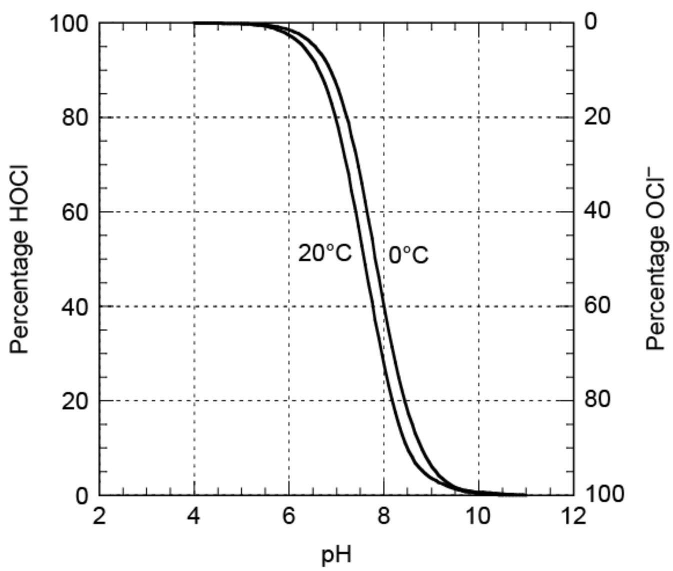


Figure 10-2
Distribution of hypochlorous acid (HOCl) and hypochlorite ion (OCl^-) in water as a function of pH at 0 and 20ºC (Tchobanoglous et al., 2014)

Important environmental and water quality characteristics that influence chlorine demand, or the amount of chlorine needed for adequate disinfection, are sunlight, inorganic species, ammonia concentration, and organic content. Chlorine solutions are photosensitive because sunlight provides the energy needed for chlorine to react with water, which is the main reason why swimming pools add chlorine frequently and bleach is sold in opaque bottles. Some inorganic and organic constituents in water react with and consume chlorine via oxidation-reduction (redox) reactions. Certain organic species react with free chlorine to form chlorinated phenols and trihalomethanes, known as disinfection byproducts because of their toxicity.

10.1.1 Chlorine Chemistry

Reactions with ammonium consume chlorine species and form chloramines, or combined chlorine and, as shown on Fig. 10-3, have a characteristics reaction sequence known as a breakpoint curve. A Cl_2:NH_4^+-N ratio of approximately 5 by weight (mole ratio of 1.0) yields the maximum concentration of combined chlorine. With the use of additional chlorine beyond this point along the breakpoint curve, chloramines are destroyed while chloro-organic compounds such as disinfection byproducts persist. If the ratio of chorine to ammonium exceeds 7.6 by weight (mole ratio of 1.5), the breakpoint is reached. With the use of additional chlorine after breakpoint, free chlorine is formed. When sufficient chlorine is reacted with ammonium such that a free chlorine residual remains, the term used to describe the process is breakpoint chlorination.

Chloramine species include monochloramine (NH_2Cl), dichloramine ($NHCl_2$), and nitrogen trichloride (NCl_3), which react with hypochlorous acid as shown in Eqs. (10-3) through (10-5). The relative presence of each species is dependent on the Cl_2:NH_4^+-N ratio, pH, and temperature. $NHCl_2$ is about 35 times more effective than NH_2Cl for bacterial inactivation and is the dominant species at lower pH values. For example, at a pH of 5.0 the fraction of $NHCl_2$ is 0.84. However, free chlorine as HOCl is over 400 times more effective at killing *E. coli* than NH_2Cl, and 11 times more effective than $NHCl_2$.

$$NH_3 + HOCl \rightarrow NH_2Cl + H_2O \qquad (10\text{-}3)$$

$$NH_2Cl + HOCl \rightarrow NHCl_2 + H_2O \qquad (10\text{-}4)$$

$$NHCl_2 + HOCl \rightarrow NCl_3 + H_2O \qquad (10\text{-}5)$$

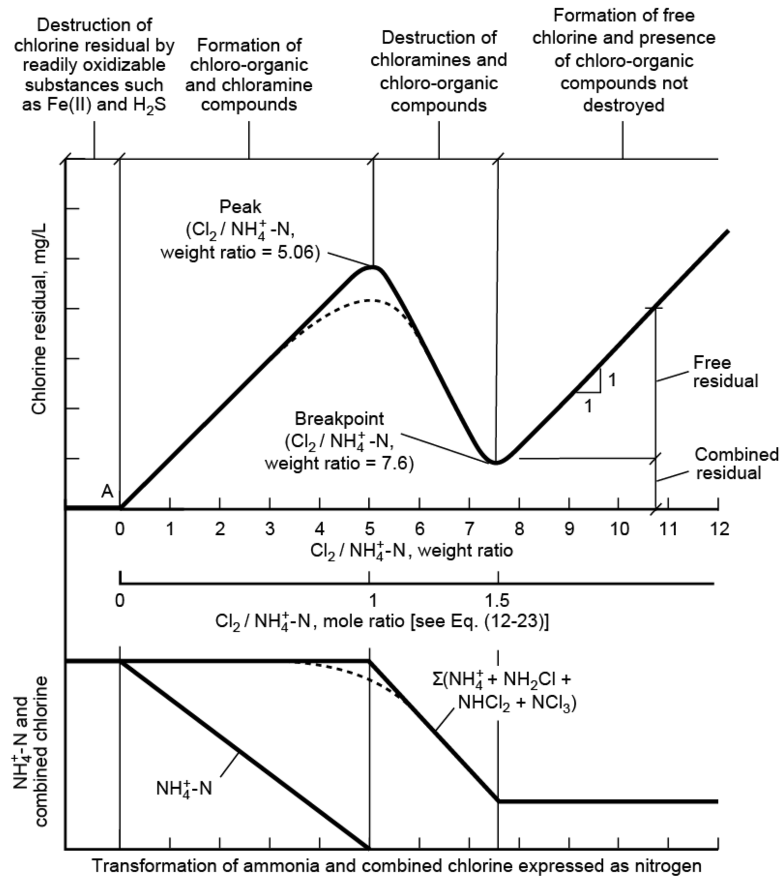


Figure 10-3
Generalized breakpoint chlorination curve.
The upper portion of the diagram represents residual chlorine as a function of the amount of chlorine added to wastewater containing ammonium. The lower portion represents the fate of ammonium and chloramines during the breakpoint chlorination process. The dashed line reflects the fact that, along with the formation of chloramines, some destruction of the chloramines occurs simultaneously before the peak is reached. (Tchobanoglous et al., 2014)

10.1.2 Impact of Water Quality

A large amount of coliform removal takes place in the treatment system upstream of the chlorination system. As shown on Fig. 10-4, in previous sampling total coliform was removed fully before flow entered the chlorination system. The primary goal of adding chlorine is to provide a residual disinfectant in the water that control biofilm growth in the plumbing and biofouling of the flush valves.

As discussed above, chlorine is a strong oxidizing agent and will react with constituents other than pathogens in wastewater. The amount of chlorine used to oxidize readily oxidizable inorganic constituents such as iron (Fe^{2+}), manganese (Mn^{2+}) and hydrogen sulfide (H_2S) comprises the initial demand. The further reaction of chlorine with (1) organic constituents and (2) ammonium forms chloro-organic compounds and combined chlorines, respectively. With further chlorine addition, combined chlorines are themselves oxidized until breakpoint is reached. Note that because of the formation of acid during breakpoint reactions, 14.3 mg/L of alkalinity is needed for every 1.0 mg of NH_4^+-N oxidized during breakpoint chlorination. This acid formation is described by Eq. 10-6.

$$2NH_4^+ + 3Cl_2 \rightarrow N_2 + 6HCl + 2H^+ \tag{10-6}$$

Breakpoint chlorination may not be necessary to achieve the required 5-log bacteriophage inactivation because this may be achieved by ozonation. Breakpoint chlorination may also not be desirable in this recycled water scheme because of buildup of TDS. Increased TDS is also observed with breakpoint chlorination at a rate of 7.1 mg/L TDS per mg NH_4^+-N consumed when sodium hypochlorite is used. However, the regrowth of coliform bacteria in the system with a low combined chlorine residual will need to be monitored.

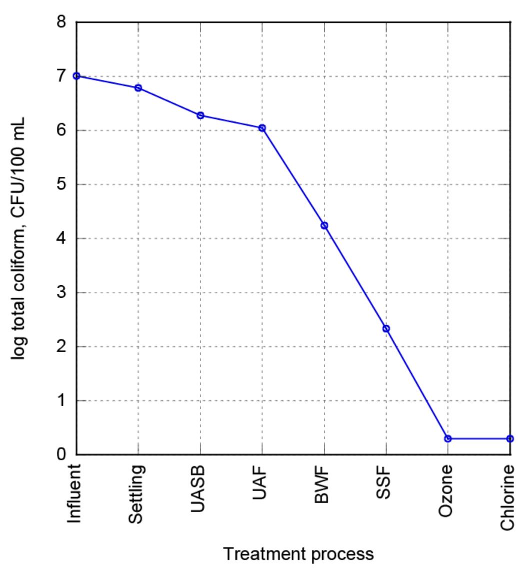


Figure 10-4
Removal of total coliform with incremental treatment steps

10.1.3 Chlorine Contact Facilities

Chlorine injection equipment is shown on Fig. 10-5. Sodium hypochlorite is stored in a bulk storage tank and pumped directly into the effluent line from the chlorine dose tank. A peristaltic chlorine injection pump is used for chemical metering. As a practical consideration, dilution may be required for applications with low chlorine demand to allow for adequate metering pump run times. Chlorine contact tanks of equal volume are arranged in series as seen on Fig. 10-1 to promote plug flow hydraulics. Total chlorine and oxidation-reduction potential (ORP) are monitored as described in Sec. 13.

(a) (b)

Figure 10-5
Chlorination system showing (a) typical chlorine metering pump and injection, and (b) chlorine contact tanks

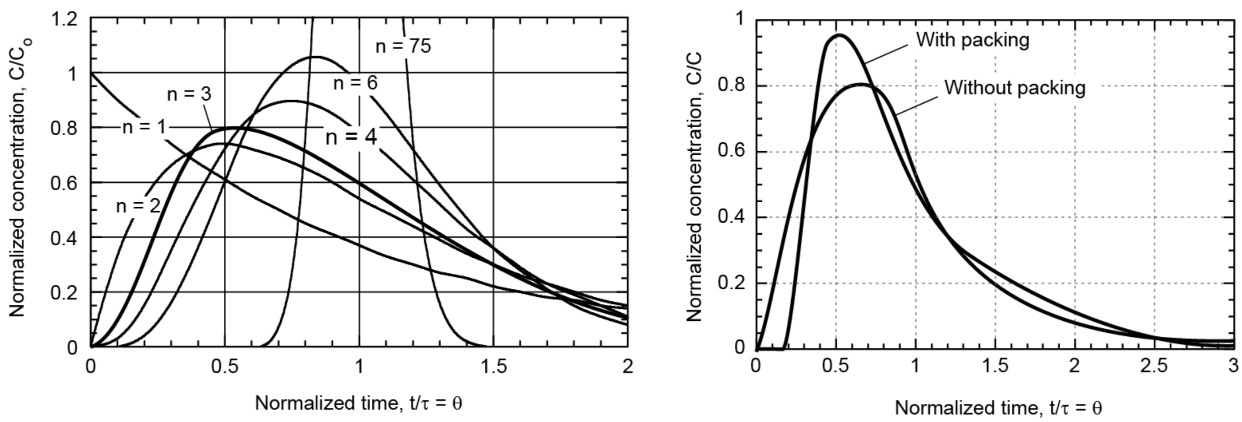
It is recommended that chlorine contact tanks provide rapid initial mixing followed by plug-flow hydraulics. Contact time is related to configuration of the chlorine contact tank (Marske and Boyle, 1973). While open channel plug flow reactors are the standard technology used for chlorine contact, two other options exist that are more suitable for the low-flows observed at SRRA facilities: (1) tanks in series, and (2) pipeline. In the tanks-in-series model, the flow moves from one tank to the next such that the system approximates plug flow. In the pipeline model, a section of large diameters (e.g., 12 to 24 inches) is used as an actual plug flow reactor, with flow in one end and out of the other.

10.2 DESIGN PROCEDURE

The design of chlorination facilities for water reuse at SRRA facilities based on chlorine demand is governed by several key factors: water quality, type and concentration of chlorine applied, contact time, pH, and temperature.

10.2.1 Modal Contact Time (T)

The modal contact time is the length of time, expressed in minutes, that water is retained in the chlorine contact tanks at peak flow conditions. The modal contact time can be determined using a tracer test to characterize the hydraulics of the particular chlorine injection system. The tracer, typically rhodamine WT or fluorescein, is injected and then effluent samples are collected to determine the residence time distribution. Model and actual tracer curves are shown on Fig. 10-6. The tracer curve obtained at the Dunnigan SRRA (see Fig. 10-6b) is indicative of poor performance because much of the water exits the CCTs before the required retention time.



(a) (b)

Figure 10-6
Analysis of retention time:
(a) models of retention time for tanks in series, and
(b) tracer curve from the Dunnigan SRRA pilot facility CCT

As shown on Fig. 10-6a, the model contact time for three tanks in series is 50 percent of the theoretical contact time determined as volume divided by flowrate. The curves on Fig. 10-6a can be used to determine design values, but tracer tests are recommended for evaluation of actual facilities prior to startup.

The modal contact of time of 90 min is required by CCR Title 22 based on peak dry weather design flow. However, it is clear, based on the curves shown on Fig. 10-6a, that effective chlorination cannot be achieved using mixed tanks in series as investigated at the Dunnigan SRRA pilot facility. The major problem is short circuiting of a significant fraction of the flow with minimal retention time. In preliminary tracer testing, a portion of the tracer started to exit that tank about 30 min after the tracer was injected.

The Dunnigan SRRA pilot facility CCBs were intended to operate as upflow tanks, approximating a plug-flow model rather than a complete-mix model; however, the high head pump used to transfer flow into the CCBs created a turbulent flow regime that resulted in substantial mixing and flow short circuiting. Strategies to improve the hydraulic regime include the addition of a diffuser or manifold to introduce flow uniformly across the cross-section of tanks, the addition of a plastic packing to diffuse the flow energy, or the use of a pipeline-model with flow diffusers. The tanks used at the Dunnigan SRRA pilot facility (see Fig. 10-5b) have a 16-inch opening and are too small to install a flow diffuser system. In addition, insects were found to colonize in the tanks through vents in the tank lids. The use of covered, open-top tanks, with a large height to diameter ratio, is recommended for future installations where chlorination is being considered.

10.2.2 Residual Concentration (C_R)

The C_R value is chlorine concentration measured in the effluent from the chlorine contact tanks. It is important to know about the form of chlorine that is expected prior to specification of monitoring equipment. If ammonium is present in the water discharged to the chlorination system, chloramines will be present unless enough chlorine is added to reach the chloramine breakpoint. If ammonium is not present or the breakpoint is reached, then free chlorine may be present in the effluent.

The form of chlorine will determine the type of equipment used for monitoring the C_R value. Examples of monitoring equipment used for online chlorine residual measurements are shown on Fig. 10-7. Addition information on chlorine analyzers is summarized in Sec. 10-3.

Figure 10-7
Alternative chlorine residual monitoring systems:
(a) electrode from Electro-chemical Devices (ECD), and
(b) colorimetric analyzer from Hach

10.2.3 Concentration Time (C_RT)

The C_RT concept as shown in Eq. (10-7) is used to ensure that adequate disinfection is achieved.

$$C_RT = \text{(residual concentration, mg/L) (modal contact time, min)} \quad (10\text{-}7)$$

To meet the disinfected tertiary wastewater standards in CCR Title 22 for recycled water, the chlorine disinfection system must maintain a minimum C_RT value of 450 mg-min. Use of the C_RT concept for sizing chlorination facilities is presented in Ex. 9.

10.2.4 Estimating Chlorine Dose

The amount of chlorine that needs to be added is used to estimate the amount of storage needed in the chlorine feed tank and to determine preliminary setpoints for chlorine dosing equipment. For example, at the Dunnigan SRRA pilot facility, the chlorine demand due to chlorine decay was found to be around 10 mg/L. Therefore, the amount of chlorine added is equal to the sum of the instantaneous chlorine demand, chlorine decay while retained in the contact tank, and the design chlorine residual. The chlorine decay during retention in the contact tank is related to the chemical reactions in the bulk water. A preliminary estimate of the required chlorine residual is given in Eq. (10-8).

$$\frac{N}{N_o} = \left(\frac{C_R T}{b}\right)^{-n} \tag{10-8}$$

where

- N = number of organisms remaining after disinfection, MPN/100 mL
- N_o = number of organisms present before disinfection, MPN/100 mL
- C_R = chlorine residual remaining after contact time t, mg/L
- T = model contact time, min
- n = slope of inactivation curve
- b = value of x-intercept when $N/N_o = 1$

Typical values for the coefficients n and b are 2.8 and 4.0, respectively, for coliform type organisms.

The design procedure for chlorine disinfection system design is summarized on Chart 10-2, and demonstrated in Ex. 9.

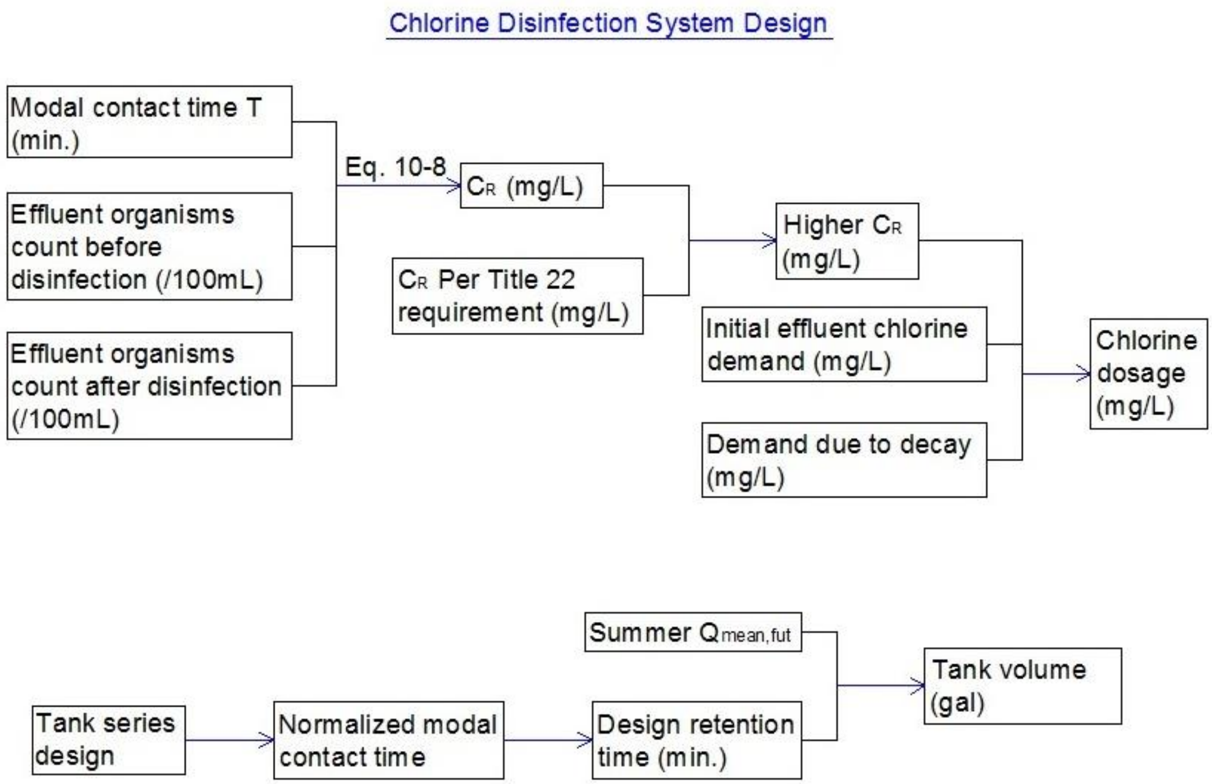


Chart 10-2
Chlorination system design flow chart

Example 9: Design chlorination facilities.

Estimate the required chlorine dose for partially nitrified effluent assuming the following conditions:

- Effluent total coliform count before disinfection: 10^5/100 mL
- Required effluent total coliform count: 1/100 mL
- Initial effluent chlorine demand: 4 mg/L
- Demand due to decay during chlorine contact: 10 mg/L
- Required modal chlorine contact time: 90 min
- Tanks in series = 3

Provide the contact volume needed and estimate the bulk sodium hypochlorite storage volume.

Solution

1. Estimate the required chlorine residual.

   Use the typical values given above for the coefficients.

   a. Summer

$$\frac{1}{10^5} = \left(\frac{C_R T}{4.0}\right)^{-2.8}$$

$$\frac{1}{10^5}^{-1/2.8} = \left(\frac{C_R T}{4.0}\right)$$

$$(61)4 = C_R\,(90)$$

$$C_R = 2.7 \text{ mg/L}$$

2. The required chlorine dosage is

   $\text{Chlorine dosage} = 4 \text{ mg/L} + 10 \text{ mg/L} + 2.7 \text{ mg/L} = 16.7 \text{ mg/L}$

3. Estimate the C_RT value for the C_R value determined in Step 2, and compare with the C_R value required by Title 22.

   a. For the proposed conditions:

      C_RT = (2.7) (90) = 244 mg/L·min

   b. Required by Title 22:

      C_R = (450 mg/L·min) / (90 min) = 5 mg/L

   c. Required chlorine dosage:

      Chlorine dosage = 4 mg/L + 10 mg/L + 5 mg/L = 19 mg/L

4. Estimate the volume of tanks required to provide a modal contact time of 90 min using three packed tanks in series.

   a. From Fig. 10-5a, the normalized modal time for three tanks in series is 0.5; therefore, the theoretical retention time for the tanks in series is:

      Retention time = (modal contact time) / (normalized modal time) =

      90 min / 0.5 = 180 min

   b. Using the design future summer mean flow (2783 gal/d), the required tank volume is:

      Tank volume = (future summer mean flow) (design retention time) =

      (2783 gal/d) (180 min) / (1440 min/d) = 348 gal

Divided into three basins and assuming the tanks are 75 percent full:

Use three 150 gal tanks (minimum)

5. Estimate the amount of chlorine used in a month.
    a. Chlorine is available at 12.5 percent, which is 125,000 mg/L
    b. At an estimated usage rate of 19 mg/L, the daily usage is:

    Chlorine use = (2782 gal/d) (3.785 L/gal) (19 mg/L) / (125,000 mg/L) = 1.6 L/d or 48 L/mo

Comment

The analysis presented in this example is preliminary in nature because it was not verified through field tests during the Dunnigan SRRA pilot study. As with all disinfection systems, this design will need to be validated in the field per SWRCB requirements. In addition, the chlorine demand may have a seasonal variability that will need to be considered during regular operation.

10.3 OTHER CONSIDERATIONS

There are many types of chlorine analyzers on the market that have different needs for calibration, operation, and maintenance. Several units are summarized on Table 10-1.

Table 10-1
Comparison of chlorine residual analyzers

		Total chlorine analyzer model			
Item	Unit	TCA-22	CCS120	CL17	TCL 11-30
Manufacturer		ECD	E&H	Hach	Rosemount
System cost	$	4350	4800	3300	5400
Replacement parts and reagent cost	$/yr	400	500	1000	3000
Type		Electrode	Electrode	Reagent	Reagent and electrode
O&M		Replace membrane annually Replace electrolyte every 4 to 6 mo. Calibrate pH and Cl sensors monthly	Replace membrane cap and electrolyte every 3 to 6 mo. Calibrate pH and Cl sensors monthly	Monthly replacement of reagents (SIRR plan for automatic ordering) Replace tubing assembly every 3 to 6 mo. Clean colorimeter once per month Replace pumps as needed (~2 yr)	Bi-monthly replacement of reagents Clean membrane every couple weeks Replace membrane assembly every 3 mo. Replace reagent and sample tubing as needed Replace fill solutions every three months Replace pumps as needed
Moving parts		Feed pump	Feed pump	Internal pumps	Internal pumps
Calibration method		1 point based on DPD	1 point based on DPD	Not needed normally	1 point based on DPD
pH and temperature compensation		pH and temp corrected	pH and temp corrected	N/A	N/A
Cl range	mg/L	0.5 – 20	0.1 – 10	0 – 5	0 – 20
pH range	unitless	4 – 12	4 – 9	5 – 8	N/A
Temperature range	degree F			41 to 104	
Sample delivery system needs		10 gal/hr feed pump @ atm	13 gal/hr feed pump @ <14.5 psi	300 mL for 30 s every 2.5 min @ 1 psi	Feed pump included
Waste flow	L/d	N/A	N/A	150	22
Location		Irvine, CA	Greenwood, IN	Loveland, CO	Irvine, CA
Phone		949.336.6060	800.478.1002	800.227.4224 x5515	909.289.2919
Contact		Jerry Berger	JPR	Gary Doty	Jeff Fittz

10.4 OPERATION, MAINTENANCE, AND TROUBLESHOOTING

The operation, maintenance, and troubleshooting requirements for chlorination facilities and related equipment such as instrumentation, internal pumps, and chemical feed systems are presented in Table 10-2.

Table 10-2
Operation, maintenance, and troubleshooting of chlorine disinfection system for recycle water at SRRA facilities.

Maintenance item	Frequency	Time, h	Tools / materials	Estimated annual cost, $[a]
Add chlorine	Monthly	0.5	Chlorine	675
Calibrate Cl sensors	Monthly	1	Field colorimeter	1320
Service sensors	Annual	2	Rebuild kits	500
Flush instrumentation panel	Quarterly	4	Chlorine	1650
Replace instrumentation pump lead tube	Annual	0.25	Replacement tube	25
Replace chlorine pump lead tube	Annual	0.25	Replacement tube	25
Service feed pumps	As needed			

a. Estimated costs include labor and materials. Labor costs estimated assuming a rate of $100/hr.

11.0 RECYCLED WATER SYSTEM DISTRIBUTION

The water that is produced by the recycled water system finally needs to be stored and pressurized for reuse in the building. The system used for storage and pressurization is known as the recycled (or nonpotable) water distribution system. Included in this section is a consideration of the layout of the recycled water distribution system and specific operational items.

This section describes the approach for designing a recycled water distribution system, as shown in Chart 11-1.

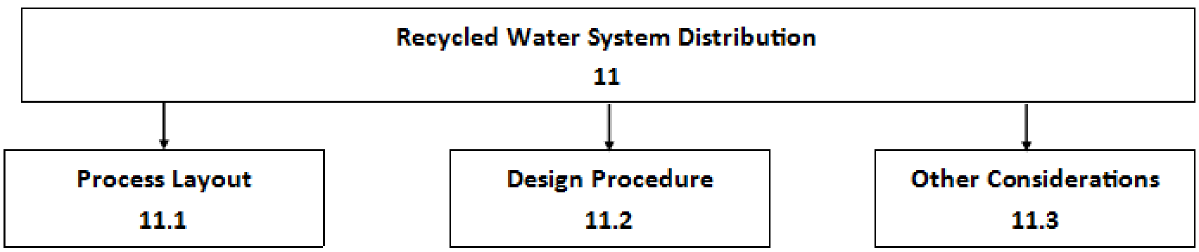


Chart 11-1
Recycled water distribution system design

11.1 PROCESS LAYOUT

Recycled water distribution systems are nearly identical to conventional booster pump systems that are used for potable water supply systems. The key difference between conventional potable and nonpotable systems described in this manual is the addition of a makeup water fill system to add water to the PWST in the event that a low water condition occurs because of the process is off-line or a peak demand. The primary components of the recycled water distribution system include the CWST, a simplex or duplex booster pump system, a pressure tank, a recycled water pipeline, and indoor non-potable plumbing. An example layout of a recycled water distribution system is shown on Fig. 11-1.

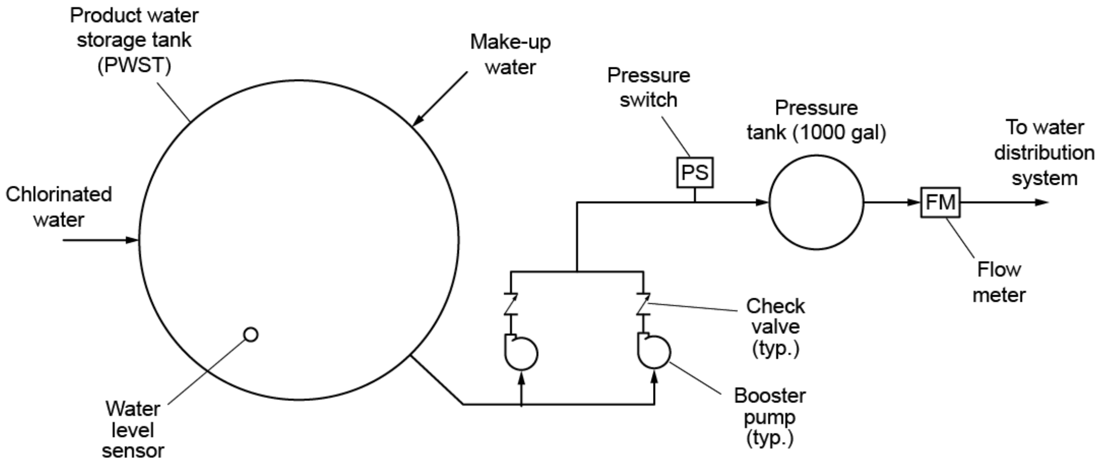


Figure 11-1
Major components of recycled water distribution system (schematic diagram)

Because booster pumps will be used to deliver the recycled water to the urinal and/or toilet fixtures, the recycled water booster pump and pressure tank can be located in a remote area relative to the comfort station. At the Dunnigan SRRA, underground directional drilling was used to install a 2-inch polyethylene pipeline about 300 ft from the comfort station. In areas that have an existing dual-plumbed water system, options for connecting to the existing water system should be explored to minimize costs associated with new pipeline installation and to ensure that no cross-connections occur.

(a) (b)

Figure 11-2
System for delivering water to indoor fixtures:
(a) booster pumps, and (b) pressure tank

It is important to ensure that all plumbing is labeled in accordance with applicable standards (see plumbing code for guidance). Plumbers and maintenance staff will need to be trained to be familiar with the issues surrounding the dual distribution systems and the importance of avoiding cross-connections between the potable and nonpotable water systems.

Flowmeters are recommended to allow for monitoring of the total water balance for the facility. In general, flow meters should be placed on the potable, nonpotable, and makeup water systems to allow for tracking real-time water use. An analysis of the flow data can be used later to fine tune the recycled water system and determine the overall water savings that were achieved.

11.2 DESIGN PROCEDURE

Based on an analysis of historical flow data at the Dunnigan SRRA, sizing the PWST about equal to the summer mean flow will result in reuse of about 88 percent of the water used in the comfort station (nearly 100 percent of the potable demand).

A few options are available for process control when the PWST reaches its maximum capacity, including (1) the treatment process goes offline until there is a demand for recycled water, (2) the PWST overflows to an irrigation or residual water dispersal system, or (3) the PWST overflows to the primary or BTPT. The specific design will depend on site-specific considerations. In general, it may be desirable to keep the system operational by allowing the PWST to overflow back to the primary tank. Maintaining flow through the system will keep the microbial community active in the primary tank and the BWF units. Allowing some overflow from the BTPT can be used to purge accumulated salts from the system.

A makeup water system is used to add water to the PWST in the event that water is being used faster than it can be processed by the BWF and SSF units. In general, the BWF units and SSF can be subjected to an increased loading to produce recycled water at a faster rate when an increased demand is detected; however, the addition of makeup water on occasion will introduce fresh water to the system and will tend to dilute accumulated salts in the recycled water. Although there are no known significant positive or negative impacts of the dilution of accumulated salts in the system, the dilution of salts will prevent chloride-based corrosion.

The indoor plumbing system and flush valves should be monitored for corrosion on a regular basis. While not anticipated, there is an increased chance of corrosion in some plumbing systems because of an elevated chloride concentration and potential for reduced pH. The type and extent of corrosion depends on the type of pipe material used and interaction with the water chemistry. Because of the complexity of metal corrosion reactions, it is challenging to predict corrosion in advance. A skilled mechanical engineer should be consulted in advance to make recommendations for protecting the plumbing for a specific facility under the expected conditions.

Makeup water should be supplied through an air gap into the PWST. The makeup water line is also expected to have a backflow prevention or check valve inline. The makeup water system is actuated when the water level in the PWST reaches a low level. Water will be added until the tank reaches a prescribed water level setpoint (for example, 500 gal). In some cases, it may be possible to use water collected from the roof of the comfort station as the makeup water supply in the nonpotable water system. The advantage of using roof runoff is that this water contains low TDS and would therefore reduce the concentration of TDS in the recycled water system.

The pressure tank should be set at a lower pressure than the potable water system so that in the unlikely event of a cross-connection, nonpotable water is not pushed into the potable water system. Additionally, a backflow prevention device should be installed if needed. Proper pipe labeling will make cross-connections an improbable occurrence.

The design procedure for PWST design for recycled water system balance is summarized on Chart 11-2, and demonstrated in Ex. 10.

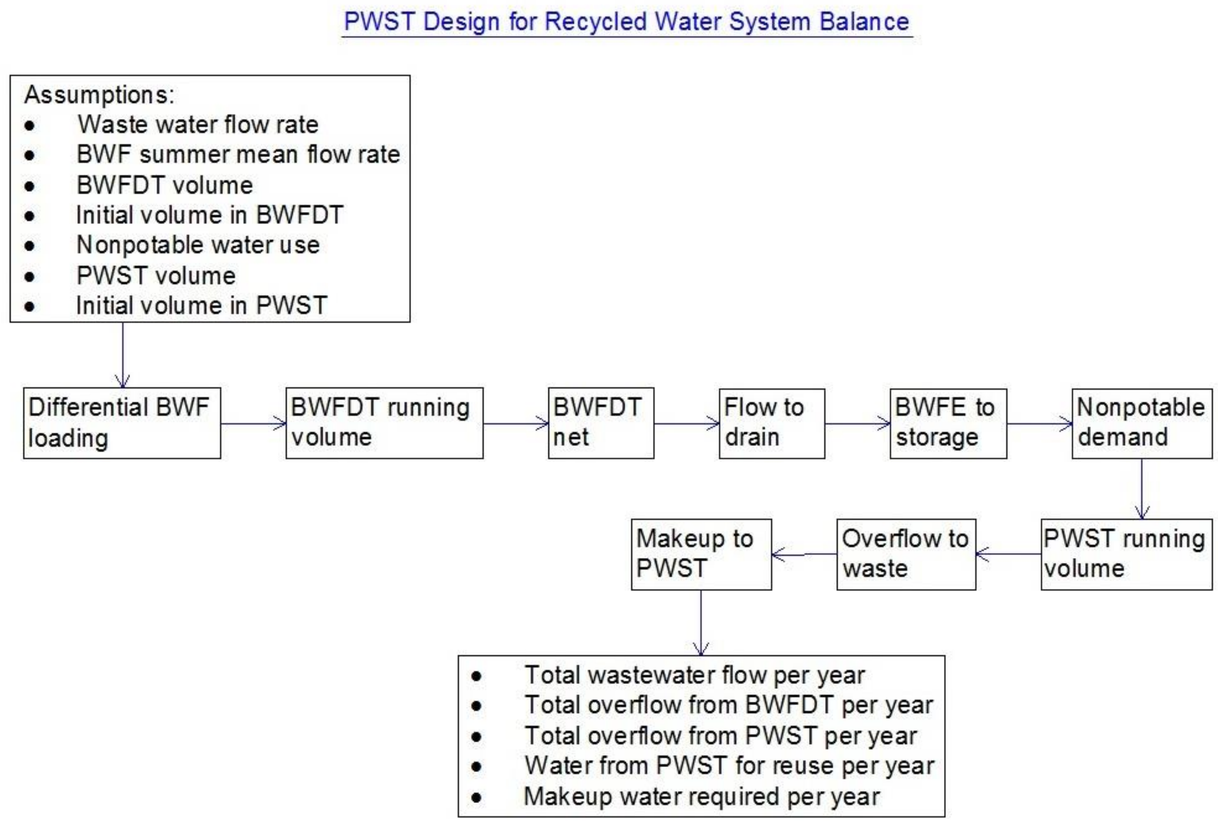


Chart 11-2
Recycled water distribution system design procedure

Example 10: Design the PWST for recycled water system balance.

Use the results in the previous examples to design the PWST for delivery of recycled water. Set up a spreadsheet to assist in computations. Apply the following assumptions in developing a solution:

- BWF flowrate = Future summer mean flow = 2783 gal/d
- BWFDT volume = 3000 gal/d
- Initial volume in BWFDT = 2000 gal
- Nonpotable water use = 92 percent of total indoor water use
- PWST volume = 3000 gal
- Initial volume in PWST = 2000 gal

Using a data set of annual flowrates, estimate the total wastewater flow, the total overflow from the BWFDT, total overflow from the PWST, water from the PWST to reuse, and amount of makeup water required.

Solution

The following computation table was developed for purposes of demonstration. The table contains data for 10 days. The full computation would require 365 days of data.

	Column j									
	(A)	(B)	(C)	(D)	(E)	(F)	(G)	(H)	(I)	(J)
Row i	Waste-water flowrate	Differential BWF loading	BWFDT running volume	BWFDT net	Flow to drain	BWFE to storage	Non-potable demand	PWST running volume	Over-flow to waste	Makeup to PWST
1	2160	-623	1377	0		2783	1987	2796	0	
2	2234	-549	828	0		2783	2055	3000	524	
3	3133	350	1178	0		2783	2882	2901	0	
4	3955	1172	2350	0		2783	3639	2045	0	
5	4250	1467	3000	817	817	2783	3910	918	0	
6	4656	1873	3000	1873	1873	2783	4284	0	0	582
7	3662	879	3000	879	879	2783	3369	0	0	586
8	1420	-1363	1637	0		2783	1306	1477	0	
9	1140	-1643	0	-6		2777	1049	3000	205	
10	2432	-351	0	-351		2432	2237	3000	195	

Steps for developing the water balance table:

1. Column (B): Compute the difference between the influent wastewater flow and the loading to the BWF.
   Differential BWF loading = Wastewater flowrate – BWF flowrate =
   2160 gal – 2783 gal = - 623 gal
2. Column (C): Compute the running volume in the BWFDT.
   a. Initial BWFDT volume = 2000 gal
   b. BWFDT running volume = BWFDT volume (i-1) + Differential BWF loading (i) =
      2000 gal + (- 623 gal) = 1377 gal
   c. BWFDT cannot exceed 3000 gal
   d. BWFDT cannot be negative

3. Column (D): Compute BWFDT net change.

   BWFDT net = BWFDT volume (i-1) + Differential BWF loading (i) - BWFDT volume (i)

   BWFDT net = 2000 gal + (- 623 gal) – 1377 gal = 0 gal

4. Column (E) = Wastewater flow to drain

   Overflow from the BWFDT = Column D if value in column (D) is positive

5. Column (F) = BWF effluent to PWST

   a. Equal to the BWF flowrate = 2783 gal, when Column (E) is positive

   b. Equal to BWF flowrate + BWFDT net change, when Column (E) is zero of negative

6. Column (G) = Compute nonpotable demand

   Nonpotable demand = [Wastewater flowrate] (fraction nonpotable water use) = (2160) (0.92) = 1987 gal

7. Column (H) = PWST running volume

   a. Initial PWST volume = 2000 gal

   b. PWST running volume = PWST volume (i-1) + BWF effluent to storage (i) – nonpotable demand (i) = 2000 gal + 2783 gal - 1987 = 2796 gal

   c. PWST volume cannot be negative

   d. PWST volume cannot exceed 3000 gal

8. Column (I) = Overflow from PWST

   a. Equal to zero when

   b. When PWST running volume is greater than PWST volume:

      Overflow = PWST volume (i-1) + BWF effluent to storage (i) – nonpotable demand (i) – PWST volume

9. Column (J) = Makeup water to PWST

   a. Equal to zero when the PWST running volume is greater than zero.

   b. When PWST is less than zero:

      Makeup = nonpotable demand (i) - PWST volume (i-1) –

      BWF effluent to storage (i)

10. Total wastewater flow = 988,955 gal/y (from spreadsheet)

11. Total overflow from the BWFDT = 66,622 gal/y (from spreadsheet)

12. Total overflow from the PWST = 39,446 gal/y (from spreadsheet)
13. Water from the PWST to reuse = 924,333 gal/y (from spreadsheet)
14. Makeup water required = 25,951 gal/y (from spreadsheet)

Comment

The recycled water distribution system is expected to supply about 93 percent of the indoor water supply. The remaining 7 percent overflows from the BWFDT. The spreadsheet can also be used to conduct a sensitivity analysis around tank size.

11.3 OTHER CONSIDERATIONS

An important consideration is how to differentiate the use of potable and nonpotable water for both the public and maintenance staff or contractors. As shown on Fig. 11-3, signs are placed above the fixtures to identify the water source. It would also be appropriate to note that water savings of 1 Mgal/y for each facility are expected for each comfort station.

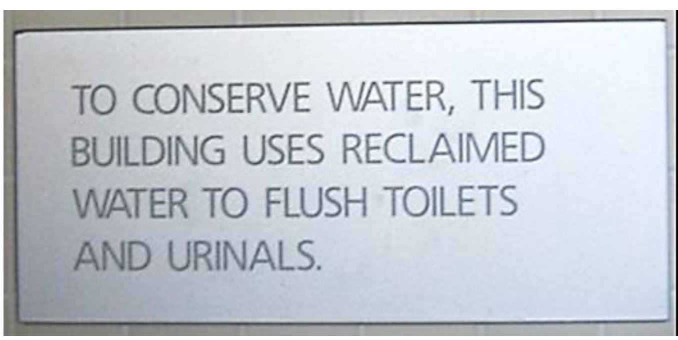
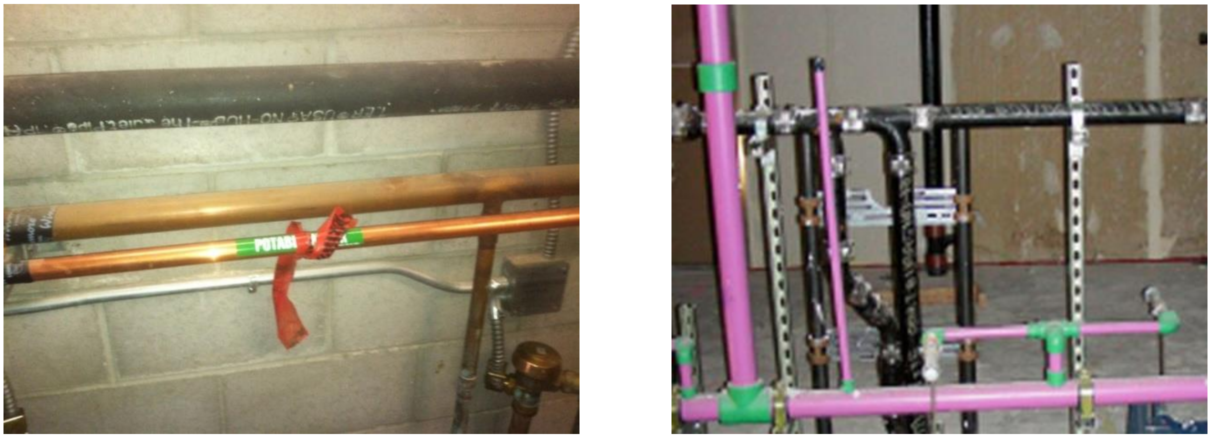


Figure 11-3
Signs placed above fixtures to notify users

The pipes in the plumbing gallery need to be identified clearly to avoid inadvertent cross-connections. Per current regulations, nonpotable lines must use purple pipe or purple tape as an indicator of a nonpotable water supply. Examples of pipe labeling are shown on Fig. 11-4.



(a) (b)

Figure 11-4
Labeling pipes to identify potable and nonpotable uses:
(a) marking with labels and flags, and (b) painting pipes with nonpotable water purple

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12.0 RESIDUAL WATER DISPERSAL

There is always extra water entering the wastewater system from sinks and floor drains that was derived from a potable source; therefore, it is expected that a a small amount of flow will need to be discharged on a regular basis. On peak days, or when there is a severe plumbing leak, there could also be more flow than can be managed with the recycled water process. Further, there will be times when the recycled water system is offline because of regular maintenance or a process/component failure. Because of these considerations on the flow balance, there is a need to have a full residual water management system.

This section describes the approach for designing a residual water dispersal system as shown in the Chart 12-1.

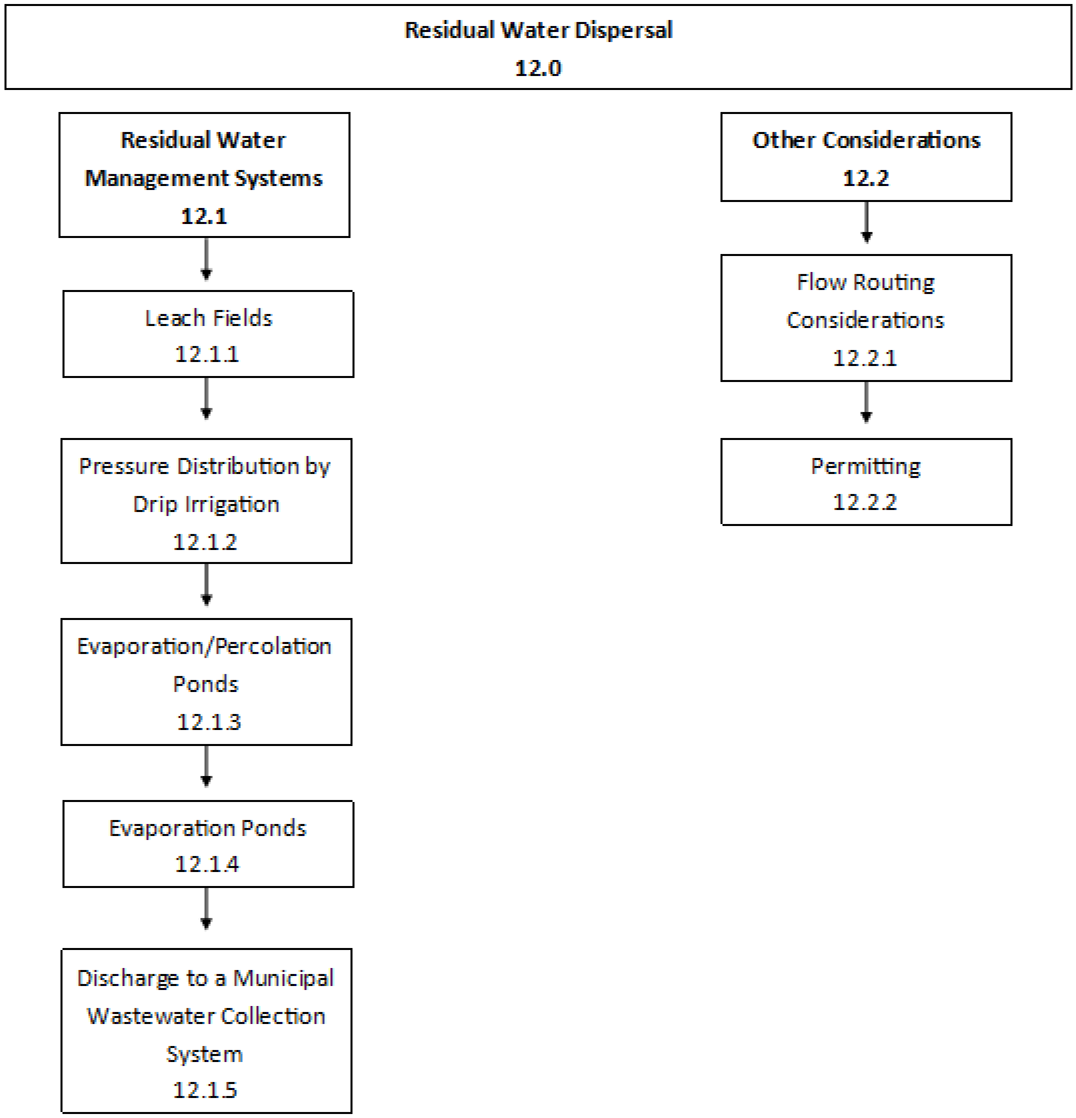

Chart 12-1
Residual water dispersal system design

Examples of residual water dispersal systems include the following.

12.1 RESIDUAL WATER MANAGEMENT SYSTEMS

Most SRRAs have existing infrastructure components that could be utilized for residual water dispersal, which are referred to here as legacy systems. These systems are typically consist of standard septic tanks and leach fields and are sized for very large flows occurring over long durations. While the legacy systems may be antiquated in terms of septic treatment and dispersal, these assets may still be able to play a large role as a component of a complete recycled water system. The advantage of these assets is twofold; they have physical capacity to accept residual water for long durations; and they are typically associated with a permitted discharge for the site. Below is a brief description and example of how these assets could be used for residual water dispersal including:

- Leach fields,
- Pressure distribution by drip irrigation,
- Evaporation/percolation ponds,
- Evaporation ponds, and
- Discharge to a municipal wastewater collection systems.

12.1.1 Leach Fields

One of the most common systems legacy systems available to use for recycled water dispersal is infiltration by gravity to shallow trenches (leachfield). The existing septic dispersal trenches can be connected to the residual water gravity overflow plumbing and be used to disperse excess water from peak days, maintenance activities or plumbing leaks. The residual water flows will be typically small with some infrequent larger flows from system maintenance or severe plumbing leaks. When compared to the original design flows for the legacy systems, the legacy systems, in most cases, will have the hydraulic capacity to disperse the residual water. Additionally, water from the recycled water plant will be of a much higher quality than the septic tanks tank effluent typically sent to these assets. Combined with the lower average daily volumes and waste strength than seen by the legacy systems in the past, the lower average daily residual water flows allow for significant resting of the trenches and soil.

One potential issue with this approach is related to the potential for high salt concentrations in the residual water. Regulatory agencies typically write requirements for constituents in a discharge in terms of concentrations. Salts will be constantly introduced into the system from the users and these salts are not removed in the treatment processes. This leads to increasing concentration with each pass of the recycled water through the system. It should be noted that, while the concentration is increasing in the recycled water product, the amount of salt over a specified time period, is the same regardless if the salts are carried in the original conventional septic system.

12.1.2 Pressure Distribution by Drip Irrigation

A variation on the leach field system is pressure distribution with drip dispersal tubing. This option can be part of an original legacy system or as a retrofit over an existing dispersal field or a new dispersal area. The advantages and disadvantages with this approach are similar to the dispersal trench described above.

There are a few things to note about using drip dispersal for residual water management. The first is related to the main difference between trench and drip dispersal. Drip dispersal, unlike trenches, does not have storage capacity in the dispersal component so management of peak flows is an area that needs to be considered. Additionally, drip dispersal is typically used in areas where turf or other plants can be utilized. The high salt concentrations in the residua water may have an impact on existing plants or plants proposed in the area. Consultation with a landscape architect should be considered to ensure the correct plant palette is selected.

12.1.3 Evaporation/Percolation Ponds

Evaporation/percolation ponds are another common legacy asset serving SRRAs. Evaporation and percolation ponds are partially lined ponds allowing wastewater to percolate into the soil as well as be evaporated into the atmosphere. Like the leach fields, these systems have the capacity to handle the much lower average flows and short-term peak flows seen from the residual water management system. Evaporation/percolation pond systems will have the same salt concentration considerations and strategies as the leach trenches. These systems have the ability for retrofitting with the addition a containment liner in the pond, converting it to an evaporation pond. Note that a water balance will have to be performed to determine what the capacity of these types of ponds would have without their ability to percolate. The evaporation pond is discussed below.

12.1.4 Evaporation Ponds

Evaporation ponds are a variation of the evaporation/percolation pond where the dispersal is only through evaporation. These ponds are fully lined to preclude any percolation into the environment. One advantage of these ponds is that the lined pond has the ability to retain the salts in the pond. The pond or pond cells can be dried in the summer months and the accumulated salts can be removed and hauled to an approved disposal facility. These solids may be considered biosolids. Disposal of biosolids is regulated by the United States Environmental Protection Agency (EPA) under Title 40 of the Code of Federal Regulations Part 503 (Part 503) and by the state under Title 22 of the California Code of Regulations (Title 22). Specific parameters regulated include metals, nutrients, pathogens, vectors, and hydrocarbons; landfill disposal typically includes analysis of metals and hydrocarbon, while land application includes analysis for metals, pathogens, and nutrients. Regional disposal facilities will have specific acceptance criteria for waste entering their facilities. A survey of the regional facilities and their acceptance criteria should be part of the design.

12.1.5 Discharge to a Municipal Wastewater Collection System

Discharge to a local municipal collection system may be an option in some locations. However, most SRRAs are located in remote areas where a municipal collection system may not be present. If this option is available and is to be explored in the discovery phase of a project, many technical and nontechnical considerations must be investigated. These include conveyance method via a force or gravity discharge line; easement and property acquisition to the closest point of connection, acceptance criteria for waste entering the municipal system; and whether the SRRA is in the district or sphere of influence, connection fees, etc.

12.2 OTHER CONSIDERATIONS

In addition to onsite management, there are several other considerations in the development of residual water management systems, including hydraulic issues and system permitting.

12.2.1 Flow Routing Considerations

The proposed location for discharge of excess flow from the recycled water system is from the BWFDT. This component of the flow has received primary treatment and therefore should contain low organic matter and particulate matter, and should be denitrified. Other process tanks should also be connected to the dispersal system for operation and maintenance purposes, including the BTPT and PWST. Implementation of these residual water management systems should be through gravity overflows, so that there is no chance for a tank to overflow or an actuated valve that could fail.

12.2.2 Permitting

Under some conditions, supplemental treatment may be required to meet groundwater discharge regulations. The appropriate regulatory agency will need to be consulted to determine the level of treatment required for the site under consideration. Because the design of onsite dispersal systems varies widely for specific sites, these systems are not considered further in this manual. However, note that most SRRA facilities have legacy wastewater systems that may be suitable to manage these residual and excess flows. In most cases, the use of the residual water dispersal system will be intermittent and at much lower flows than that of the legacy systems' original design flow. This is an important point when navigating the permitting process.

13.0 PROCESS MONITORING AND CONTROL

Monitoring techniques, including manual and online methods and control systems used for manual and remote operations, are discussed in this section, as shown on Chart 13-1.

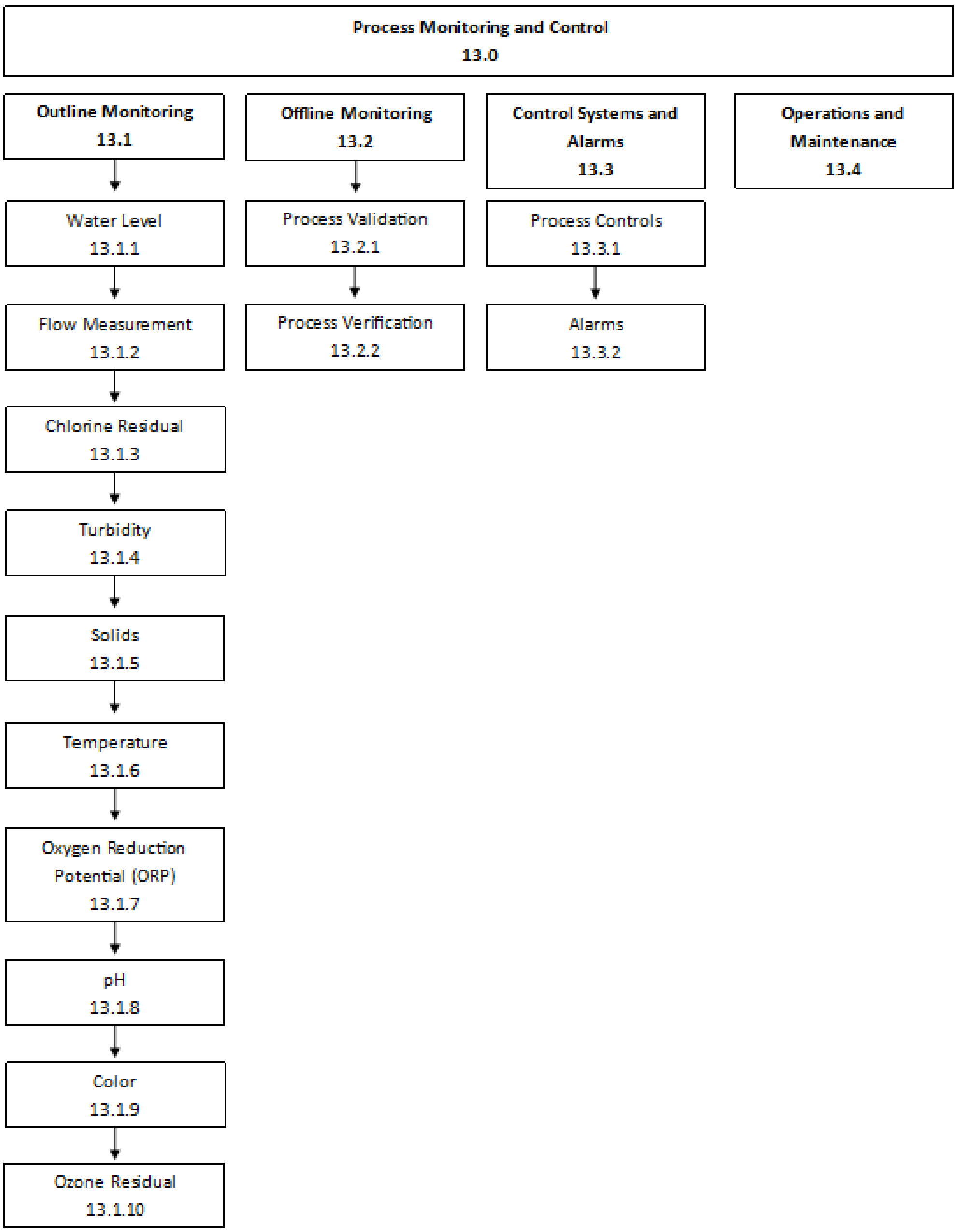

Chart 13-1
Monitoring techniques

Monitoring locations are shown on the process flow diagram on Fig. 13-1. Online instrumentation as well as key process control points are indicated on the figure and referenced below.

13.1 BASELINE MONITORING

A significant amount of background data have been collected in a process known as baseline monitoring at the Dunnigan SRRA and other SRRA facilities throughout the state. These baseline data are used to establish the background for the risk assessment. Because of the high degree of similarity among all SRRA facilities, baseline monitoring data can be composited for use in developing the risk model. To date, baseline monitoring has been limited to total and fecal coliform concentrations at various steps in the treatment process. Additional baseline monitoring data are needed for other pathogens such as virus, protozoa, and helminth microorganisms.

13.2 VALIDATION MONITORING

Following commission and startup, it is expected that a period of time will be required for development and stabilization of biological processes. After the treatment system reaches steady-state operation, challenge testing should be used to validate the design and performance. Proposed challenge tests could include the filtration, ozone, and chlorine systems. Process validation may include the use of dye tracers, microbial surrogates (i.e., MS2 coliphage), total cell counts, particle size distribution, and total organic carbon. The validation strategies will need to be documented in the engineering report submitted to the SWRCB with the permit application.

13.3 OPERATIONAL MONITORING

A SCADA system will be used to monitor and control the treatment train remotely, as described below. Each of the parameters below is monitored via online instrumentation and controlled with a remote SCADA system.

13.3.1 Water Level

Water levels are monitored in the product water storage tank, the SSFs, the biological treatment process tank, the solids processing/upflow sludge contact tanks, and the grinder pump lift station. The product water storage tanks are supplied by both recycled product water (PW)and potable make-up water. The product water booster pumps are called on when there is demand. When the water level in the product water tank drops below a given setpoint, the make-up water valve is actuated automatically to maintain this low water level (LWL) setpoint. There is also a high water level (HWL) setpoint above which the product water tank goes into overflow mode.

Hydrostatic head builds up on the SSFs as clogging develops. If the water level increases above a normal operating point, the float switch indicates an alarm condition to the PLC and the filter enters overflow mode. Overflow mode consists of a gravity line into the primary tank or BWFDT, where a gravity overflow to dispersal is provided.

To identify the filter or filters in need of maintenance, the plant operator assesses effluent flow rates from each filter in the field using the rotameters on each filter effluent line. Low flow from a filter or a large head differential in a filter indicates filter clogging. The operator takes each clogged filter offline for maintenance. During this period, the flow is sent to the other filters or can be manually diverted to dispersal. Note that during maintenance, the SSFs are drained to the SSFDT to avoid dispersal of the large volume of water in each filter; instead, the filter drain water is stored during maintenance and re-applied to the operational filters.

The grinder pump lift station should be equipped with water level sensing equipment. Various types of water level sensing equipment have been used, including ultrasonic level sensors, pressure transducers, and float switches. Because of problems with the operation of float switches in systems with high solids and the potential for scum accumulation, float switches are not advised for routine water level detection in raw wastewater lift stations. Instead, a combination of systems are recommended as follows: (1) routine water level sensing for purposes of pump control should be done by ultrasonic detection or pressure transducer type system, and (2) float switches should be used for the redundant off/low level alarm and high water alarm.

13.3.2 Flow Measurement

Monitoring locations for flow rates, including flow meters and pump run times, are listed in Table 13-1. Where pump run times are monitored, pump curves and (if possible) field validation of flow rates are used to estimate flows. Where flow is needed to determine process loading rates or chemical feed rates, for example, flow meters are used. The flow meter readings are processed with the SCADA system for real-time water balance information, including nonpotable water use, potable water use, overflow water, make-up water, and internal recycle flows.

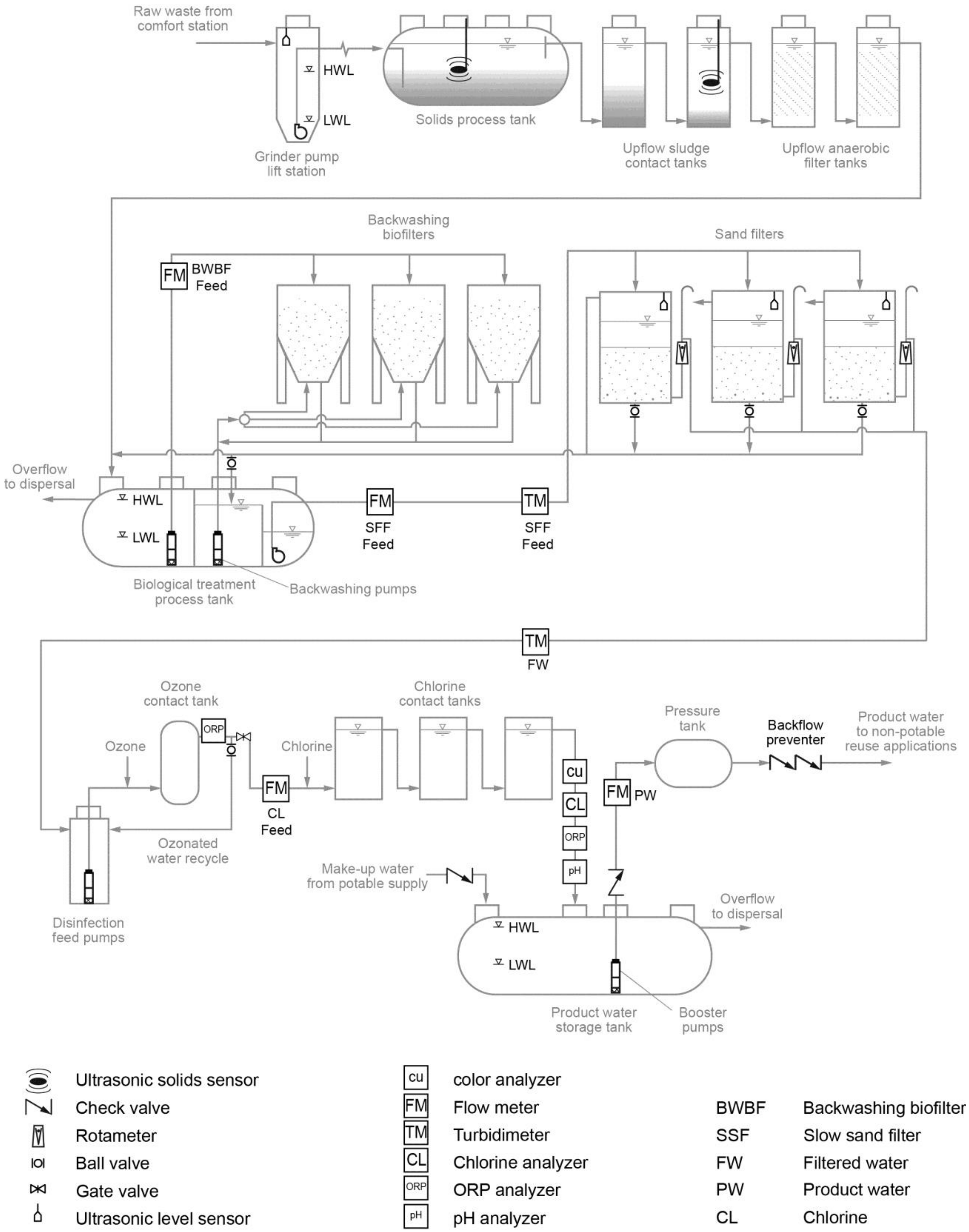

Fig. 13-1
Locations of monitoring and controls (process flow diagram)

Table 13-1
Flow meter and pump run time monitoring locations

Location	Flow Measurement: Pump run time	Flow Measurement: Flow meter	Comment
Lift station grinder pumps (2)	X		Water level control
Scum suppression pump	X		Optional; timer control
BWF dosing pumps (2)		X	Water level control; the difference between BWF feed flow and SSF and disinfection feed flows equates to total backwash flow
SSF feed pumps (2)		X	Water level control; used to verify loading rate to filters
BWF backwash pumps (2)	X		Frequency of backwash pre-programmed and adjustable from SCADA
Ozone feed pumps (2)		X	Water level control; used to flow pace chlorine feed
Chlorine metering pumps	X		Run times set to target chlorine residual
Product water booster pump (2)		X	Pressure switch control to maintain pressure in pressure tank based on system demand

13.3.3 Chlorine residual

Chlorine is monitored for both chemical consumption (dose) and residual concentration. The chlorine dose is determined by C_RT requirements and modal contact time. The chemical metering pump run times are programmed to meet the specified dose and chemical usage is monitored via pump run times as well as chlorine scales.

To ensure that chorine is always available to the chlorine metering pumps, a chlorine level detection and standby chlorine supply should be included as part of the chlorine dosing skid.

The residual concentration is monitored at the chlorine contact basin effluent point. Total chlorine is measured by an amperometric ion-selective electrode with temperature compensation. An alarm activates when an analyzer or sensor fault occurs. The residual chlorine concentration is used by the SCADA system to calculate the CrT value based on a manually entered modal contact time as determined from tracer testing.

Should the total chlorine residual drop below the setpoint (e.g., 10 mg/L), an alarm condition will be indicated via the SCADA. This alarm condition will trigger the actuated valve to divert water from the product water storage tank.

13.3.4 Turbidity

Turbidity is monitored at the slow sand filter (SSF) influent and SSF effluent. A SSF influent turbidity greater than 5 NTU that persists for 15 minutes or more actuates a diversion valve to the existing dispersal system. A SSF effluent turbidity greater than 2 NTU (daily average) will also activate the diversion valve per CCR. Turbidity analyzers are connected to the SCADA system, which performs continuous averaging to determine whether the influent or effluent turbidities are in excess of these trigger points. The actuation of the diversion valve is controlled via SCADA; an alarm will sound locally and a master alarm will be triggered on SCADA.

13.3.5 Solids

The accumulation of solids is measured in the solids process tank and upflow sludge contact tanks via the solids monitoring system. An ultrasonic sensor and transducer assembly suspended in the water column indicates levels of scum and sludge. The sensor reads a distance to the bottom of the scum and to the top of the sludge. Based on programmed tank dimensions and normal liquid level, sludge and scum thickness measurements are recorded in a local control unit (CU) and uploaded to the SCADA system periodically.

Data are recorded on the CU every minute for the most recent hour (MRH), every hour for the most recent day (MRD), and every day for the most recent month (MRM). Records stored over a month are kept as a single monthly data summary (MDS); all data are averaged over the given time period. SCADA is programmed to query the CU hourly or as needed.

When a predetermined percentage of the tank is filled with solids, the solids monitoring system will send an alarm to the SCADA system. Alarms can also be programmed for a given scum or sludge thickness. If the liquid level exceeds a given range outside of a programmed normal liquid level, an alarm will also be triggered. Temperature is also monitored by the solids monitoring system (unalarmed).

13.3.6 Temperature

Temperature will be monitored in the solids process tank and upflow sludge contact tanks as well as in the diurnal flow equalization tank to better anticipate performance of biological processes. Temperature measurement is provided through the solids monitoring system, as described in that section. A temperature sensor is provided in the diurnal flow equalization tank such that loading to the backwashing filters can be adjusted accordingly.

13.3.7 Oxygen Reduction Potential

ORP is an indication of the efficacy of the ozonation process. The required ozone dose will fluctuate with water quality. When a higher ozone dose is required than that applied, the ORP in the ozone or chlorine contact tank effluent will decrease. An ORP sensor probe can be used to monitor this variability and a minimum ORP setpoint will be programmed into the process controls so that an alarm will be triggered when the ORP drops below this setpoint for a given period of time.

Should a low ORP alarm occur, the operator will have the ability to increase the recycle ratio of the ozonation process by increasing the run time of the ozone recycle pump. The ozone recycle pump run time can be changed via the local control panel or remotely via SCADA.

13.3.8 pH

A low pH can cause corrosion in the distribution system, which contains copper pipes and brass fittings known to be reactive in acidic conditions. Therefore, pH will be monitored in the disinfection effluent prior to distribution. While the treatment train is expected to maintain adequate buffering capacity to maintain a pH of at least 6.3, controls can be used to adjust the system alkalinity should the pH decrease below a given setpoint.

A portion of the backwashing filter nitrified effluent is recycled to the solids process tank via the backwash return piping system. The nitrified effluent is denitrified in the solids process tank and upflow sludge contact tanks, which recovers some alkalinity and ultimately increases the disinfection effluent pH. A low pH, as determined by a programmed setpoint, will trigger an alarm on the local control panel (LCP) and SCADA.

13.3.9 Color

The use of color removal as a surrogate for virus inactivation is a concept described in Sec. 9. Additional information will be provided pending the results of further testing. Online photometric color analyzers are commercially available and are expected to provide a viable strategy for process monitoring.

13.3.10 Ozone Residual

Similar to chlorine residual, ozone can be measured to obtain target ozone residual values. However, in the Dunnigan SRRA pilot study, in almost all cases, ozone residuals were not detected because the ozone generator equipment was undersized for the demand. For future systems, it is recommended that a properly sized ozone system be used and ozone monitoring equipment be considered.

13.3.11 Field Observations

The plant operator or service provider will perform some field monitoring of water quality parameters and process testing to ensure proper operation. These field monitoring techniques are summarized as process validation and verification testing.

13.4 PROCESS VERIFICATION

Process verification equates to field sampling of selected parameters that are indicative of proper system operations. Process verification could take place at regular intervals, such as daily, weekly, or monthly, or may be required if there is a process modification or after some period of being offline. The proposed process verification strategies will be documented in the engineering report submitted to the SWRCB with the permit application. These measurements may include bacterial sampling or other water quality parameters and will need to be conducted in accordance with the permit. Process verification could also include periodic challenge testing of the ozonation process using bacteriophage MS2, an influenza virus protein, or similar.

13.5 CONTROL SYSTEMS AND ALARMS

The control system will include logic that either modifies process operations to correct an out-of-spec scenario or activates an alarm or service request.

13.5.1 Process Controls

The output from the online monitoring equipment will be used to monitor data trends for process control. The data trend monitoring will focus on parameters that can be used to assess process stability. The following parameters have been identified as important for use in characterizing the integrity of the treatment process:

- Chlorine dose – The dose is the amount of chlorine that must be added to achieve a suitable chlorine residual concentration.
- Residual total chlorine at the end of the chlorine contact basin – This parameter is used to ensure that CRT requirements are being met.
- Residual total chlorine in the product water storage tank – Chlorine in the water that is delivered to the plumbing system must contain chlorine to prevent biofouling in pipe networks.
- Turbidity in the feed to the slow sand filter – This parameter is used to indicate upstream process integrity and to track the development of clogging in the SSF.
- Turbidity in the product water – Low turbidity in the slow sand filter effluent is essential for achieving disinfection

- pH in the product water – Various processes are impacted by pH, including nitrification, chlorination, and corrosion. Tracking system pH will make it possible to detect and respond to variations in process operations.
- Color in ozonated water – As described in Sec. 9, correlations between color removal and virus removal were established at the Dunnigan SRRA pilot study facility.

Chlorine and turbidity analysis indicate whether the treatment system is meeting the requirements set forth in CCR Title 22 for recycled water. The pH analysis provides an indicator of the biological process performance.

13.5.2 Alarms

Alarms can be programmed to alert the plant operator and/or Caltrans local maintenance station staff. The SCADA system will allow personnel to make changes to setpoints, review and acknowledge alarms, and track current and previous plant data. Security for the SCADA system generally is set up so that all authorized personnel can track plant performance and review current or previous alarms while requiring passwords to acknowledge alarms and change setpoints. The passwords are used to ensure that only qualified personnel make changes to setpoints or acknowledge alarms. Unique passwords are typically given to each person so that the SCADA system can track who made changes and acknowledged alarms. The SCADA system can be set up to notify the proper personnel of current alarms through phone calls, text messages, or emails.

Alarms are provided with an independent power supply (e.g., battery) for the following conditions:

- Loss of power from the normal power supply
- Failure of a biological treatment process (e.g., turbidity, pH)
- Failure of a disinfection process (e.g., color, disinfectant residual, out of chemical)
- Failure of a filtration process (e.g. turbidity, high water level)

- Any other specific process failure for which warning is required by the regulatory agency

Emergency effluent management is provided via an overflow diversion to the existing onsite dispersal system if the plant is shut down or the lift station is offline.

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